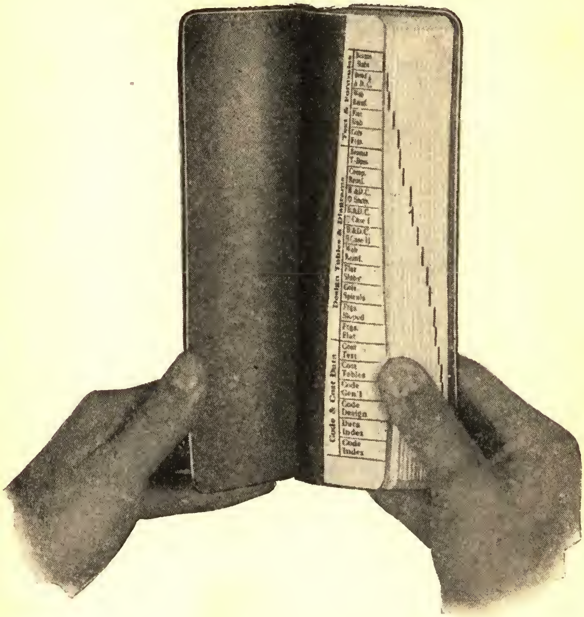
HANDBOOK
OF
REINFORCED CONCRETE BUILDING DESIGN

Use of the Marginal Index



A heavy black slug is placed on the margin of the first page of each section dealing with a separate subject. By bending the book backward these slugs are exposed opposite the individual section headings, which are printed on this page.

If the thumb of the right hand is placed on the slug opposite the section desired, the book can be opened directly to the corresponding text, tables or diagrams.

Text & Formulas	Beams Slabs
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Design Tables & Diagrams	Beams T-Bms
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Code & Cost Data	Cost Text
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	Code Design
	Data Index
	Code Index

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A HANDBOOK OF
REINFORCED CONCRETE BUILDING DESIGN
IN ACCORDANCE WITH THE 1928
JOINT STANDARD BUILDING CODE

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Price, $1.00
First Edition
September, 1928

Authorized Reprint
from the copyrighted Proceedings
Volume 24—1928
AMERICAN CONCRETE INSTITUTE
DETROIT, MICHIGAN

Reprinted By
PORTLAND CEMENT ASSOCIATION
CONCRETE REINFORCING STEEL INSTITUTE
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ACKNOWLEDGMENT

THE paper "Design and Cost Data for the 1928 Joint Standard Building Code" and the report of Committee E-1 on "Reinforced Concrete Building Regulations and Specifications" are authorized reprints from the copyrighted Proceedings, Volume 24, 1928, of the American Concrete Institute,.

The American Concrete Institute is an organization composed of engineers, architects, contractors, manufacturers and others whose object is to provide a means of finding the best way of doing concrete work and disseminating that knowledge. The work of the Institute is technical and educational and not of a trade or commercial character. The activities are divided among approximately thirty committees whose members are actively engaged in professional or industrial occupations closely related to the work under consideration by the respective committees. Many of the reports of the committees are developed into standards of design and practice.

The American Concrete Institute has a membership of more than twenty-seven hundred throughout the United States, Canada and many of the foreign countries. The papers, reports and discussions of the Institute are issued in a bound volume of Proceedings covering the activities of each year.

Further information as to the Institute's activities and publications may be had by addressing the American Concrete Institute, 2970 West Grand Boulevard, Detroit, Michigan.

FOREWORD

FOR many years there has been a demand for a compact yet comprehensive handbook on reinforced concrete building design so presented as to facilitate the efforts of designers and to conserve their time.

Successful performance of reinforced concrete over a long period of years together with accumulated knowledge of extensive research in concrete and reinforced concrete opens new possibilities in structural design. Stress limits based upon a required quality of concrete are now used with full confidence and safety due to improvements in construction methods and equipment and a growing understanding and application of the basic laws controlling the quality of concrete.

In presenting to the engineering profession this first edition of "A Handbook of Reinforced Concrete Building Design," its sponsors have endeavored to eliminate variables which limited the utility of former publications in this field. This handbook embodies tables and diagrams for the design of structural members of buildings in accordance with the provisions of the Joint Code. The design practice which is presented, offers a means of obtaining a degree of economy consistent with the quality of the concrete used. The maximum economy, however, will only be possible where strict control of concreting operations prevail.

The experience and training of the author of Design and Cost Data, Mr. A. R. Lord, combining a knowledge of theory and research with extensive practical experience in the design and construction of many buildings, have fitted him for this work to a remarkable degree.

A fundamental knowledge of the mechanics of structures and reinforced concrete design is essential to the intelligent use of this handbook.

PORTLAND CEMENT ASSOCIATION
CONCRETE REINFORCING STEEL INSTITUTE
RAIL STEEL BAR ASSOCIATION.

Design and Cost Data for the 1928 Joint Standard Building Code.

By Arthur R. Lord.*

SYNOPSIS.

The development of the technique of concrete proportioning within recent years, the constantly advancing knowledge of the mechanics of reinforced-concrete building design, the long years of study and research embodied in the 1924 report of the Joint Committee on Specifications for Concrete and Reinforced Concrete and the subsequent careful codification of that report by the Building Code Committee (E-1) of the American Concrete Institute has made available a workable and authoritative building code for all types of reinforced-concrete construction such as engineers in any city may adopt with confidence. One objection to such adoption lies in the loss of usefulness of most of the design tables and diagrams which have cost engineers a great deal in both time and money. To overcome this objection this paper includes a complete set of designers' tables and diagrams for use with the proposed 1928 Joint Standard Building Code. I believe that engineers will find this set of designers' aids as complete, as time-and-labor-saving and as accurate as any similar set they may be using under their local code. These tables and diagrams introduce important simplifications in the design of doubly-reinforced beams and in the spacing of stirrups. They cover a much wider range of concrete strengths than is covered by similar tables and diagrams previously published. Their use is illustrated and explained by numerous examples.

With these tables and diagrams as a foundation, a study has been made of the relative cost of the common types of structures using 2,000-lb. concrete as is now almost universal, except in columns, and using concrete of considerably greater ultimate strength. The advantage of higher strength concrete is indicated by many considerations. Our new control knowledge indicates that a fifty per cent increase in strength over the usual performance in concrete making in the past may readily be obtained with an increase in cost of about ten per cent. The use of higher strength concrete would also result in more workable concrete, less permeable and more highly resistant to the usual exposures to which outdoor concrete is subject. The attempt to utilize our newer knowledge of concrete propor-

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tioning to produce a better and a cheaper 2,000-lb. concrete is likely to result in harsh mixes and expensive placing. It seems to me better to use the same amount of cement, or somewhat more cement, with less water, and to employ in the design the higher strength so secured. The reduction in water will more than balance the addition of cement, insofar as shrinkage and crazing is concerned. This study indicates that a considerable direct saving in the final cost of an ordinary building is secured in addition to a higher quality of concrete and an easier field operation.

PART ONE—DESIGN DATA.

Introduction.—Almost from its founding, the American Concrete Institute has had a committee at work preparing and revising a building code which the Institute recommends as a standard, fairly representative of the best up-to-date knowledge and practice in reinforced-concrete design. This code is available to governmental agencies and has been adopted by a number of cities as well as by individual designers. The Institute has also been represented on the two National Joint Committees which have worked on this same problem on the *specification* side. I have been intimately connected with this work, as a member of the Institute's committee on building code for fourteen years, including five years as chairman, and as one of the Institute's representatives on the second Joint Committee. I have also participated professionally in the re-drafting of several city building codes and am now a member of the committee engaged in revising the Chicago code. The 1928 edition of the Institute's Standard Reinforced-Concrete Building Code is based on the 1924 Joint Committee report, in which the specification form has been changed to code form and in which scientifically accurate but practically cumbersome formulas and provisions have been simplified and made more workable as a basis of design and construction. In a few instances there are differences in the substance of the two reports but the greater part of the differences are entirely matters of form, adopted to make a *code* more acceptable to the engineers who must use and enforce it. In 1927 the Committee on Standard Practice of the Concrete Reinforcing Steel Institute which had previously adopted a code based on the 1924 Joint Committee report joined with Committee E-1 of the American Concrete Institute to formulate the 1928 Joint Code on which this paper is based. With respect to cost of buildings erected, this code is fairly representative of the more advanced general practice. Most city codes present certain sections which are far more liberal than the average national practice and in comparison with these sections the Joint Code would indicate added cost. On the other hand most city codes have other provisions which are unnecessarily burdensome and in these sections the Joint Code would show a saving. Comparisons with the Chicago code are included in Part II of the paper.

Some of the great differences in city codes have arisen from a feeling that the quality of the concrete in various localities must necessarily vary. We know today that a concrete of 2,000 or 3,000-lb. strength can be made

as readily in one city as in any other. Other differences in city codes have arisen from the political ascendancy of some business interest and represent merely inequitable variations in the factor of safety required in different members. The present situation in which 2,000-lb. concrete is limited to 650-lb. stress in one locality and forced to carry 1,200-lb. a few miles away, even though made from identical materials, is so absurd that the general adoption of a scientifically accurate and practically workable standard, such as the Joint Code, is a reasonable hope.

I have been using the substance of the 1928 Joint Code quite generally in my work since 1924, where city ordinances have not taken precedence, and as a result, prior to starting this paper, I had accumulated a considerable number of tables and diagrams to facilitate rapid and accurate design. I also recognized the need of still other designers' aids which time had not permitted me to develop. One great hardship involved in the adoption of the Joint Code by engineers generally would be the necessary scrapping of the old design tables and diagrams, on which much time and money had been spent, and the large new effort and expense which would be involved in creating in each office the necessary new diagrams and tables. This paper has been undertaken in the hope of supplying this need and of removing this obstacle. It represents also the conservation of a huge amount of duplicated effort which would otherwise be necessary. The preparation of the paper at this time has been made possible by aid from the Portland Cement Association, the Concrete Reinforcing Steel Institute and the Rail Steel Bar Association, making up the cost to me of producing the tables and diagrams not previously worked up and of making all of them conform strictly to the Joint Code as now presented.

Basis of the Tables and Diagrams.—In the preparation of these tables and diagrams, I have had in mind the importance of simplicity of presentation, in order to reduce both the time required for their application and the liability of error. Each diagram is based on a single steel stress and a single concrete stress, except that in some instances four parallel tables have been printed together to save space. In any practical design, only one set of stresses is involved and the work is facilitated and safeguarded if only this set is represented on the diagram. All stresses and provisions appearing in the paper correspond to my interpretation of the Joint Code. This code is unusually direct and understandable.

In the treatment of T-beams and of beams with compressive reinforcement I have adopted the device of reducing all designs to substantially the same process and method as is now universally used in the case of the rectangular beam. This method is based on the use of the full allowable stresses in both the steel and the concrete, in other words on the use of "balanced reinforcement." This has involved a large amount of computation to get accurate values of p and K for the great range of beam proportions used in design, but has greatly simplified and reduced the labor of design. By the tables presented here the usual involved diagrams are rendered unnecessary. A workable solution of the T-beam with compressive

reinforcement is presented for the first time and in the same simple manner. The unusual length of this paper has made it necessary to omit all formula derivation, but the formulas used are stated and figures are drawn in which the various dimensions and forces have been so represented as to aid anyone who wishes to check them. Every formula, table and diagram has been checked by some competent engineer other than myself and entirely outside of my own office.

In the design of certain types of members, such as footings or flat slabs, the proportions of the final structure may be varied through a considerable range and a large variety of equally correct designs may be secured, all complying with the Code. By a careful study of the average conditions encountered in practice it is possible—and customary in large offices—to set up standards of proportions to apply to all members of a given type. In such members all horizontal dimensions can be made to bear a constant ratio to the side of the floor panel or of the footing and all vertical dimensions a constant ratio to the depth of the slab or footing. In this way very simple diagrams can be prepared for "standardized" members and a vast amount of time and labor saved. This has been done in this paper and the "office standards" presented are fully described. In all such cases they are based on extended use in practice of substantially the same proportions.

The design of web reinforcement has been a thorn in the side of the concrete designer, with the result that wasteful guesswork has come to be all too common. The well-known shear diagram, which is easily sketched even for the most complicated cases as soon as the loads and reactions have been computed, has been used in this paper to compute directly and rapidly both the number of stirrups (vertical or inclined) and their accurate spacing. The diagrams perform for the designer the tedious and exacting work which has heretofore been required for accuracy and economy.

The work of the Division of Simplified Practice of the Department of Commerce in conjunction with the various trade organizations of the building industry has resulted in the elimination of much waste. In the field of reinforced concrete this work has resulted in the establishment of eleven standard bar sizes and of four standard sizes for spiral rods. Bar and spiral rod sizes which are no longer standard have been eliminated from my tables and diagrams. This will automatically eliminate the need for substitutions and back checking which would result from the accidental use of non-standard sizes, no longer commercially available. In the same way standard column capital sizes are used in one set of the flat slab diagrams.

Notation.—Standard notation as used in the Joint Code has been employed in all the text, tables, and diagrams of this paper. A few additional symbols have been used in some instances, and these are defined in the text.

General Tables.—Tables 1 and 2 are general tables, used in connection

with several types of members. They need very little explanation. Numerous other tables of this general group are available, but are not included when the results may be obtained by a simple slide rule operation and are usually so obtained by designers in preference to turning to the table.

Table 1 applies to the stem width of beams, T-beams, joists, etc., and its use will save computations involving both multiplication and addition.

Table 2 gives necessary information as to bars. Most designers know bar areas, but need to refer to such a table for values of $\sum o$ (= Summation of bar perimeters), in making bond computations. The value of $12a_s$ is useful in determining the bar spacing in one-way slab design.

Steps in Design for Flexure.—The design of all types of beams comprises seven steps, which are essentially the same for rectangular and T-beams with and without compressive reinforcement. These seven steps are stated completely under "Steps in Design of Rectangular Beams." In the other types of beams the slight differences occasioned by the introduction of special factors (t/d, d'/d, b' and p') are explained in full under each design step that is affected by them, while the steps that remain unchanged are not repeated on account of space limitations. The designer may not always be conscious of the individual steps but the process that he carries out is essentially that described below.

Steps in Design of Rectangular Beams.—(1) The size and weight of the member are assumed and the moments, shears and reactions are computed.

(2) The value of the effective depth, d, is assumed, the value of K for the concrete stress used in the design is taken from Table 3 or Table 4 and the value of b is computed by formula (101).

$$b = \frac{M}{Kd^2} \quad \dots\dots\dots\dots (101)$$

(3) The value of v, the unit shearing stress, is computed by formula (102a).

$$v = \frac{8V}{7bd} \quad \dots\dots\dots\dots (102a)$$

(4) The value of v must lie within the limits permitted by the code, which vary with the type of anchorage, and the weight of the beam including protective covering must agree with the weight assumed. If necessary, new assumptions must be made and the first four steps repeated.

(5) From the value of p for balanced reinforcement corresponding to the value of K used above, compute the area of tensile reinforcement by formula (103a).

$$A = pbd \quad \dots\dots\dots\dots (103a)$$

(6) Select bars from Table 2 to make up the required area, checking the bond unit stresses by formula (17) of the code and the necessary stem width by Table 1.

(7) Complete by designing the web reinforcement.

Problem 1 shows the complete design of a rectangular beam by this common method while Fig. 3 gives the formulas relating to p and k, on which Tables 3 and 4 are based, and illustrates the stress relations.

Many cases will arise in design when it may be advantageous to maintain a uniform size of beam for several moment conditions, rather than to preserve "balanced reinforcement." In such instances the design will usually be made for the limiting beam from these tables and the concrete stresses in the other beams will be less than the full allowable value by the code. For these beams with reduced concrete stress, the steel area will be determined by formula (103b) in place of formula (103a) above.

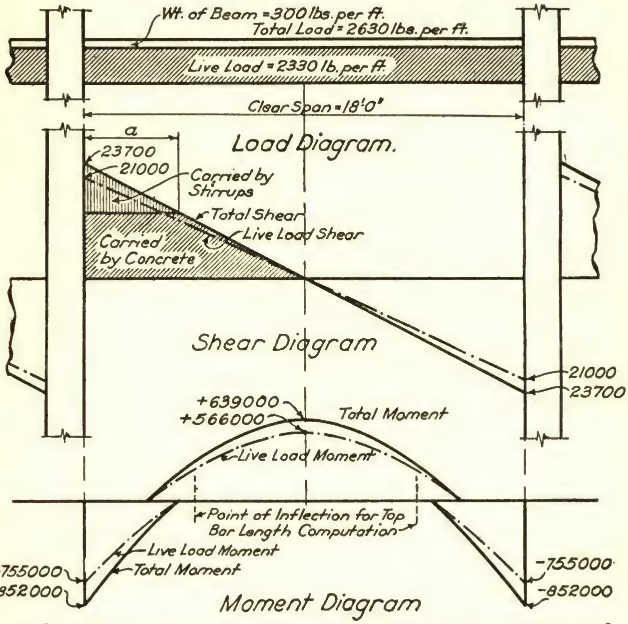

FIG. 1.—LOAD, SHEAR AND MOMENT DIAGRAMS FOR PROBLEM 1.

$$A = \frac{M}{17{,}500d} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(103b)}$$

PROBLEM 1.

Having given the load, shear and moment curves for the superimposed uniformly distributed load as shown by the dash lines in Fig. 1 complete the design of the rectangular beam in accordance with the 1928 Joint Code, using 2,000-lb. concrete, fire resistive construction and deformed bars. The beam frames into reinforced-concrete columns at each end. The adjoining spans on either side are the same as this span.

Solution: Assume a beam 12 by 24 in. in section. The weight will be $(12)(24)(150)/144 = 300$ lb. per lin. ft. For this loading:

Dead-load moment at support $= wl^2/12 = \dfrac{(300)(18)(18)(12)^{(a)}}{12} =$ 97,000 in.-lb.

Total moment at support, $M_c = 97{,}000 + 755{,}000 = 852{,}000$ in.-lb.

Dead-load moment at center $= wl^2/16 = \dfrac{(300)(18)(18)(12)^{(a)}}{16} =$ 73,000 in.-lb.

Total moment at center, $M_c = 73{,}000 + 566{,}000 = 639{,}000$ in.-lb.

Dead-load reaction at each support $= (9)(300) = 2{,}700$ lb.

Total end shear $= 2{,}700 + 21{,}000 = 23{,}700$ lb.

The moment and shear curves, revised to include the dead-load, are shown in Fig. 1 by heavy full lines.

For fire resistive construction $d = 24 - (1\tfrac{1}{2} + \tfrac{1}{2}^{(b)} + \tfrac{5}{8}^{(c)}) = 21.4$ in. for assumed beam depth.

From the total end shear and formula (102a) $v = \dfrac{(8)(23{,}700)}{(7)(12)(21.4)} =$ 106 lb. per sq. in. ($= 0.053f'_c$). This is less than 120 lb. per sq. in. ($0.06f'_c$) and requires only ordinary anchorage.

At the support, in accordance with Table 4,

$$K = 157,\ p = 0.0091$$

By formula (101) $b = \dfrac{852{,}000}{(157)(21.4)^2} = 11.9$ in. (Against 12 in. assumed—O. K.)

By formula (103a)* $A_s = (0.0091)(11.9)(21.4) = 2.31$ sq. in.

At the center, in accordance with Table 3, $K = 131$, $p = 0.0075$

$$\text{By formula (101) } b = \frac{639{,}000}{(131)(21.4)^2} = 10.7 \text{ in.}$$

By formula (103a)* $A_s = (0.0075)(10.7)(21.4) = 1.71$ sq. in.

Several combinations of bars will satisfy this requirement. The arrangement that supplies the required steel areas both over the support and at the center with the least total cost of the steel in place should be used. In this determination the extras for bar sizes, the extras for bending or cutting small amounts of any one size and the extra labor cost of handling bars of many lengths varying by small amounts should all be considered.

Try 3-⅞ in. rd. at the center, bending up 1-⅞ in. rd. bar and lapping across the support. This will provide 2-⅞ in. rd. across the support ($=$ 1.20 sq. in.) and 2-⅞ in. rd. straight in the top will make up the required 2.31 sq. in. From formula (17) of the code the bond stress on the lower steel at the point of inflection will be:

$$u = \frac{8\,V}{7\,\Sigma\,od} = \frac{(8)(14{,}200)^{(h)}}{(7)(5.5)^{(e)}(21.4)} = 138 \text{ lb. per sq. in. } (= 0.069f'_c)$$

*Formula (103b) may be used, as described on page 6.

This exceeds 100 lb. per sq. in. ($0.05f'_c$) and requires special anchorage for at least one-third of the center reinforcement. One ⅞ in. rd. must be carried into the support at each end a distance equal to $(16.7)^{(d)}$ $(⅞) = 14.6$ in. One ⅞ in. rd. bar will be carried to the center of the support. The bond stress on the upper steel at the edge of the support will be, by formula (17) of the code:

$$u = \frac{(8)\,(23{,}700)}{(7)\,(11.0)\,(21.4)} = 115 \text{ lb. per sq. in. } (= 0.058f'_c)$$

This exceeds 100 lb. per sq. in. ($0.05f'_c$) and requires special anchorage beyond the point of inflection (the fifth point of the clear span) of at least one-third of the steel. In this case the 2-⅞ in. rd. straight in the top must be carried $(16.7)^{(d)}\,(⅞) = 14.6$ in. beyond the point of inflection or 4 ft. 10 in. beyond the face of the support.

In the shear diagram (Fig. 1) the vertically-hatched area representing the shear taken by the stirrups at either end is a *triangle*, which indicates that this is Case II (see p. 22) and the rapid solution for stirrup spacing given in Table 68 is available. The shear carried by the concrete by equation (114) is:

$$V_c = (60)^{(a)}\,(⅞)\,(12)\,(21.4) = 13{,}500 \text{ lb.}$$

The distance, a, to the point where no web reinforcement is required, by formula (119) is:

$$a = \left(\frac{23{,}700 - 13{,}500}{23{,}700}\right)\left(\frac{(18)\,(12)}{2}\right) = 46.5 \text{ in.}$$

The area of the triangle under the shear curve is:

$$\Sigma\, V' = \left(\frac{23{,}700 - 13{,}500}{2}\right)(46.5) = 237{,}000 \text{ in. lb.}$$

The maximum permissible size of a vertical stirrup by Diagram 66 is ⅜ in. rd. for deformed stirrups. The total stirrup area at each end by formula (117) is:

$$NA_v = \frac{237{,}000}{(14{,}000)\,(21.4)} = 0.79 \text{ sq. in.}$$

From Table 67 we find that 4-⅜ in. rd. U-stirrups equal 0.88 sq. in.
From Table 68 for 4 stirrups and $a = 46.5$ in. we compute

Face of support to first stirrup = (.07) (46.5) = 3.25 in.	= 3¼ in.
Next two spaces = (.16) (46.5) = 7.44 in.	2 at 7½ in. = 15 in.
Last space = (.26) (46.5) = 12.1 in.	1 at 12 in. = 12 in.

By section 804, the maximum permissible stirrup spacing within the distance, a, is:

$$(0.75)\,(21.4) = 16.0 \text{ in.}$$

No extra stirrups are required by this limit. The spacing from the face of each support is:

3¼ in., 2 at 7½ in., 12 in.

The stirrup length is:

$$(12 - 3) + [(2)(21.4)] + [(2)(5)] = 62 \text{ in.} = 5 \text{ ft. } 2 \text{ in.}$$

A portion of the stirrups could be omitted in the zones where the bent-up bar reinforces the web. The omissions may be readily determined by drawing the stirrups and the bent bar to scale on the shear diagram as indicated in Fig. 12.

Notes for Problem 1:

(a) Multiplied by 12 to give moment in *inch*-pounds.
(b) Allowance made for ½ in. rd. stirrups—slightly excessive.
(c) Allowance made for 1¼ in. rd. bar—slightly excessive.
(d) By section 903 of the code:

Length of anchorage with deformed bars =

$$\frac{\left(\frac{20{,}000}{3}\right)\left(\frac{D^2}{4}\right)}{(0.05f'_c)(D)} = \frac{33{,}333D}{f'_c} = 16.7D \text{ for 2,000-lb. concrete.}$$

(e) Σo for 2-⅞ in. rd. = 5.5 sq. in. from Table 2.
(f) Σo for 4-⅞ in. rd. = 11.0 sq. in. from Table 2.
(g) Since special anchorage has been provided to meet the bond requirements the value of v_c may be taken as $0.03f'_c$ or 60 lb. per sq. in. for 2,000-lb. concrete.
(h) Shear at point of inflection (fifth point of clear span) equals (0.6) (23,700) = 14,200.

Steps in Design of T-Beams.—T-beams, in which the slab acts as a compression flange for the beam stem, are the same as rectangular beams in which the values of p and K are reduced by the elimination of part of the section. In the analysis used in this paper, the compressive stresses in the stem between the neutral axis and the lower face of the flange are neglected, as is usual in design. In designing a T-beam the thickness, t, of the slab forming the flange has presumably been determined. The steps in the design are as follows:

(1) Same as rectangular beam except that only the weight of the *stem* of the beam need be assumed.

(2) The value of d is assumed, and from this the value of t/d is at once known. Enter Table 5 under the concrete stress used in the design and locate the value of K opposite the computed value of t/d. Record the value of p for use under step (5). From formula (101) determine the value of b.

(3) From formula (102a) rewritten with b', the stem width, in place of b, compute the value of v.

$$v = \frac{8\,V}{7\,b'd} \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(102b)}$$

(4) The value of v must be checked against the limits permitted by the code and the overhanging width of the slab used for the T-flange on either side of the stem must be checked also against the code. The weight of the stem including protective covering must be checked against the assumed weight.

(5), (6) and (7) are exactly the same as in the rectangular beam.

Problem 2 shows the complete design of a T-beam by this method while Fig. 4 gives the formula relating to p and K in such a beam, on which Table 5 is based, and illustrates the stress relations.

Steps in Design of Rectangular Beams with Compressive Reinforcement.—Compressive steel is introduced into a rectangular beam when the values of b and d are so limited by architectural considerations as to make the value of K from formula (104) greater than the value given in Table 3 (or Table 4) for the concrete stress permitted in the design.

$$K = \frac{M}{bd^2} \qquad \text{.................................... (104)}$$

Since b and d are both known the design steps are modified as follows:

(1) The weight of the member is known and the moments, shears and reactions have been computed.

(2) Knowing d, the value of d'/d is closely established by the amount of covering required by the code. This should be taken as 0.02, 0.04, 0.06, etc., to 0.20 (whichever is the nearest to the value computed). Enter the appropriate table—8 to 15 inclusive—for the concrete stress used in the design and under the proper value of d'/d locate the value of K found by equation (104). Record the corresponding values of p and p' for use under step (5).

(3) and (4) are the same as in the rectangular beam.

(5) The area of the tensile reinforcement is found from formula (103a) and the area of the compressive reinforcement from formula (105).

$$A'_s = p'bd \qquad \text{.................................... (105)}$$

(6) and (7) are the same as in the rectangular beam, with the added requirement of supplying the ties for the compressive steel.

In Tables 8 to 19 for special refinement of design the change in the values of p and K for small increments of compressive reinforcement are found by interpolation, permitting any degree of accuracy warranted by the balance of the design or the number of like beams involved. Problem 2 shows the complete design of a beam with compressive reinforcement by this simple method, while Fig. 5 gives the formulas relating to p, p' and K in such a beam, on which Tables 8 to 15 are based, and illustrates the stress relations.

PROBLEM 2.

The T-beams of a beam-and-girder floor are spaced six feet apart on centers and are supported on concrete girders spaced 24 ft. in the clear. The live-load and the weight of the 4-in. floor slab total 300 lb. per sq. ft. The adjoining spans on either side are the same as this span. Design the beam both at the center and at the support in accordance with the Joint Code, using 3,000-lb. concrete and fire resistive construction. The depth of the beam is limited by architectural considerations to a maximum of 16 inches.

Solution: Assume the beam stem as 12 in. wide. The weight of the stem, below the slab, will be (12)(12)(150)/144 = 150 lb. per lin. ft. The load from the slab will be (300)(6) = 1,800 lb. per lin. ft. and the total load 1,950 lb. per lin. ft.

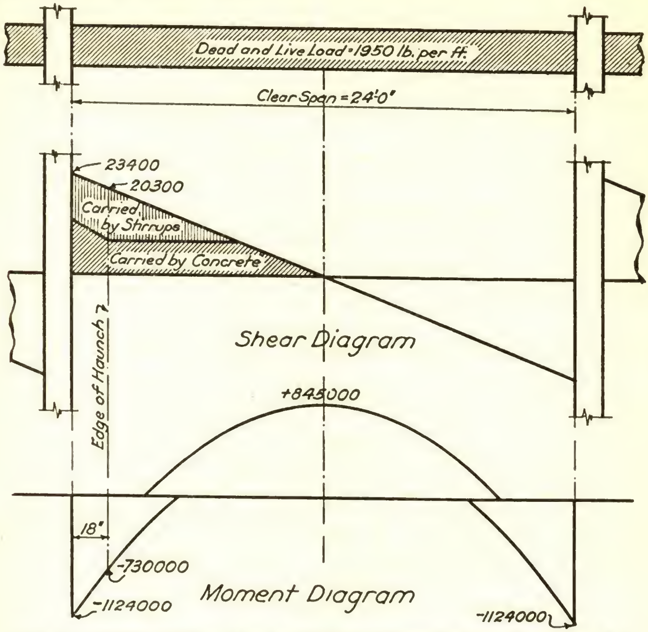

FIG. 2.—LOAD, SHEAR AND MOMENT DIAGRAMS FOR PROBLEM 2.

Design at center (T-beam).

Moment at center, $M_c = \dfrac{wl^2}{16} = \dfrac{(1{,}950)(24)^2(12)}{16} = 845{,}000$ in. lb.

$t = 4$ in. $d = 16 - (1\frac{1}{2} + \frac{3}{8}{}^{(a)} + \frac{5}{8}{}^{(b)}) = 13.5$ in.

$t/d = 4/13.5 = 0.30$

From Table 5, for 3,000-lb. concrete, $p = 0.0108$ and $K = 192$

By Formula (101) $b = \dfrac{845{,}000}{(192)\ (13.5)^2} = 24.1$ in.

By Formula (103a)* $A_s = (0.0108)\ (24.1)\ (13.5) = 3.52$ sq. in. = 2-1⅛ in. sq. and 1-1 in. sq.

From Table 1, width of beam = $9\frac{1}{2} + (\frac{3}{4}{}^{(a)} + 1^{(c)}) = 11\frac{1}{4}$ in. (12 in. assumed above—O. K.)

By formula (102b) $v = \dfrac{(8)(12)(1{,}950)}{(7)(12)(13.5)} = 165$ lb. per sq. in. $(= 0.055 f'_c{}^{(e)})$

*Formula (103b) may be used, as described on page 6.

This is less than 180 lb. per sq. in. ($0.06f'_c$) and special anchorage is not required for diagonal tension. The overhanging flange width is only 6.1 in. on each side and is O. K. Assume that 1-1⅛ in. sq. bar will be bent up and the two remaining bars carried through in bottom.

From formula (17) of the code the bond stress at the point of inflection will be:

$$u = \frac{(8)\ (7.2)\ (1{,}950)}{(7)\ (8.5)^{(d)}(13.5)} = 140 \text{ lb. per sq. in. } (= 0.047f'_c\ )$$

This is less than 150 lb. per sq. in. ($0.05f'_c$) and only ordinary anchorage is required.

Design at Support (Rectangular Beam with Compressive Reinforcement).

$$\text{Moment at support } M_s = \frac{wl^2}{12} = \frac{(1{,}950)\ (24)^2(12)}{12} = 1{,}124{,}000 \text{ in. lb.}$$

$b = 12$ in. and $d = 13.5$ in. as determined above, $d' = 2.7$ in.

$$d'/d = \frac{2.7}{13.5} = 0.2$$

$$\text{By formula (104) } K = \frac{1{,}124{,}000}{(12)\ (13.5)^2} = 513$$

An examination of Table 13 shows that K is too great and that the beam must be widened at the support. Try $b = 20$ in.

$$\text{By formula (104) } K = \frac{1{,}124{,}000}{(20)\ (13.5)^2} = 308$$

From Table 13 using $d'/d = 0.2$; for $K = 313$; $p' = 0.016$ and $p = 0.0185$.

By formula (105) $A'_s = (0.016)\ (20)\ (13.5) = 4.32$ sq. in.

If the 1-1⅛ in. sq. and 1-1 in. sq. in the bottom are lapped, an area of 4.53 sq. in. will be provided.

By formula (103a)* $A_s = (0.0185)\ (20)\ (13.5) = 5.00$ sq. in.

The 1⅛ in. sq. bent up from the center and lapped across the support provides 2.53 sq. in. Two 1⅛ in. sq. straight in the top will provide 2.53 sq. in. or a total of 5.06 sq. in.

The critical bond stress at the face of the support will be:

$$u = \frac{(8)\ (12)\ (1950)}{(7)\ (18.0)^{(f)}\ (13.5)} = 110 \text{ lb. per sq. in. } (= 0.037f'_c\ ) \text{ requiring}$$

ordinary anchorage only.

The increase of b to 20 in. at the support must be tapered down to the center width of 12 in. not nearer the support than that point where the moment is reduced to the value of the resisting moment of the doubly

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*Formula (103b) may be used, as described on page 6.

reinforced 12 by 16-in. section. This may generally be assumed from an inspection of the moment curve but may be computed as follows:

For 2—1⅛ in sq. and 2—1 in. sq. in bottom, $b = 12$ in. and $d = 13.5$ in.; $p' = 0.028$
For 4—1⅛ in. sq. in top, $b = 12$ in. and $d = 13.5$ in.; $p = 0.032$

The code permits only 2 per cent of compressive reinforcement to be considered effective and with $d'/d = 0.2$ the value of K from Table 13 is limited to 333. The maximum resisting moment of the 12 by 16 in. section is therefore:

$$M = (333)(12)(13.5)^2 = 730{,}000 \text{ in. lb.}$$

The haunch may be terminated at a point 18 in.[g] from the face of the support where this negative moment is shown by Fig. 2.

The unit shearing stress has been decreased by the increase in b. At the support by formula (102a):

$$v = \frac{(8)(12)(1950)}{(7)(20)(13.5)} = 99 \text{ lb. per sq. in.}$$

At the end of the haunch, by formula (102b).

$$v = \frac{(8)(10.5)(1950)}{(7)(12)(13.5)} = 145 \text{ lb. per sq. in. } (= 0.048 f'_c)$$

Only ordinary anchorage is required for either bond or diagonal tension and the value of v_c is $0.02 f'_c = 60$ lb. per sq. in.

By formula (114) $V_c = (60)(⅞)(12)(13.5) = 8{,}500$ lb.

At face of support $V_c = (60)(⅞)(20)(13.5) = 14{,}200$ lb.

Compute the stirrups from the narrow end of the haunch toward the center of the beam as in Problem 1 and continue the next-to-end spacing for stirrups within the haunch. Formula (119) determines the value of a and is not affected by the haunch. In this case the stirrups are more than adequate to meet the code requirements for ties for the compressive reinforcement.

Notes for Problem 2.

(a) Allowance made for ⅜ in. stirrups.
(b) Allowance made for 1¼ in. bar.
(c) Allowance made for fire resistive construction.
(d) $\Sigma\ o$ for 1—1⅛ in. sq. and 1—1 in. sq. bars = 8.5 sq. in. from Table 2.
(e) Note change in beam width and recalculation of v later in problem.
(f) $\Sigma\ o$ for 4—1⅛ in. sq. bars = 18.0 sq. in. from Table 2.
(g) May be computed as follows:

$$M_x = 1{,}124{,}000 - 730{,}000 = 394{,}000 \text{ in. lb.} = 32{,}800 \text{ ft. lb.}$$
$$= \frac{wx}{2}(l - x) = 975x(24 - x)$$

Solving: $x = 1.50$ ft. = 18 in.

Steps in Design of T-Beams With Compressive Reinforcement.— T-beams may occasionally require compressive reinforcement in order to comply with the code limitations of b and still keep within an architec-

tural limitation on the beam depth or flange width. This case is slightly more complex but still simple under this method of solution. The limiting values of b, d and t/d will be known. The steps are as follows:

(1) Same as for ordinary T-beam, except that weight of beam stem is definitely determined.

(2) From the values of b and d compute the required value of K by formula (104). Enter Table 16 (or Table 17, 18 or 19 according to the concrete stress used in the design) and record the value of p and K for the known value of t/d, using the three left-hand columns of the table. Subtract this K, which is the value for a T-beam without compressive reinforcement, from the total K found from formula (104). The remainder is the value of K which must be added by the compressive reinforcement and its balancing additional tensile reinforcement. Under the proper value of d'/d locate the value of K equal to this remainder and record the corresponding values of p and p'. Add the two values of p together to get the total percentage of tensile reinforcement for the beam.

(3) Same as ordinary T-beam.

(4) Same as ordinary T-beam except that the values of b and of the weight need not be checked.

(5) Compute the area of tensile reinforcement from formula (103a) and of the compressive reinforcement from formula (105).

(6) and (7) are the same as in the rectangular beam. In Tables 16 to 19 the change in the values of p and K for small increment of compressive reinforcement may be computed readily by proportion, permitting very accurate interpolation where conditions warrant.

Problem 3 shows the complete design of a T-beam with compressive reinforcement by this simple method. Fig. 6 gives the formulas relating to p, p' and K, on which Tables 16 to 19 are based and illustrates the stress relations.

PROBLEM 3.

A floor has standard pans 8 in. deep by 20 in. wide and 2-in. concrete top slab. The clear span between joist supports is 20 feet.

Live Load = (2.08) (50)	= 104
Top Slab = (2) (25)	= 50
Joist Stem = $\left(\frac{5+7}{2}\right)$ (8)	= 48
Susp. Ceiling = (2.08) (10)	= 21
Wood Floor on Fill = (2.08) (20)	= 42
Partitions = (2.08) (35)	= 73
Total Load per lin. ft.	= 338

The loads on the typical joist are as given to the left, and the design provides joists 5 in. wide and 25 in. on centers. A storage room requires a number of joists to be designed for a live load of 250 lb. per sq. ft., the other loads remaining as before except that the partition load may be considered as included in the heavier live load. The depth of the construction must remain 10 in. as before for architectural effect on the ceiling of the room below. Design these special joists, using compressive reinforcement if necessary, with 2,000-lb. concrete.

Solution: For the typical joists the design at the center is summarized as follows:

$$d = 10 - (1^{(a)} + \tfrac{1}{4}^{(b)} + \tfrac{1}{2}^{(c)}) = 8.25 \text{ in. } t/d = \frac{2}{8.25} = 0.24$$

From Table 5, for 2,000-lb. concrete, $p = 0.0065$ and $K = 117$

$$b = \frac{(338)(20)^2(12)}{(16)(117)(8.25)^2} = 12.7 \text{ in. } A_s = (0.0065)(12.7)(8.25) = 0.68 \text{ sq.}$$

in. = 1-¾ in. rd. and 1-½ in. sq.

Width of joist from Table 1 $= 4^7/_{16} + \tfrac{1}{2}^{(b)} = 4^{15}/_{16}$ in. (5 in. used).

The loading on the store room joists will be increased by (250) (2.08) = 520 lb. per lin. ft. and decreased by (2.08) (85) = 177 lb. per lin. ft., the net increase being 343 lb. per lin. ft. and the final load on these joists 681 lb. per lin. ft. For this load:

$$b = \frac{(681)(20)^2(12)}{(12)^{(d)}(117)(8.25)^2} = 34.3 \text{ in.}$$ This is greater than the available width of 25 in. and compressive reinforcement is required.

Design of T-Joist with Compressive Reinforcement

$$\text{Moment at Center} = \frac{(681)(20)^2(12)}{12^{(d)}} = 272{,}400 \text{ in. lb.}$$

$$b = 25 \text{ in. } d = 8.25 \text{ in. Therefore } K = \frac{272{,}400}{(25)(8.25)^2} = 160$$

From Table 16, left portion, with $t/d = \frac{2}{8.25} = 0.24$, $K = 117$ and $p = 0.0065$.

The deficiency in K is therefore $160 - 117 = 43$

$$d' = 1^{(a)} + \tfrac{1}{4}^{(b)} + \tfrac{3}{8}^{(c)} = 1\tfrac{5}{8} \text{ in.}; \quad d'/d = \frac{1.63}{8.25} = 0.2$$

From Table 16, right portion, using $d'/d = 0.2$ we find that $K = 42$ when $p' = 0.010$ and $p = 0.0026$.

The total percentage of tensile steel is $0.0065 + 0.0026 = 0.0091$.

By formula (103a)* $A_s = (0.0091)(25)(8.25) = 1.88$ sq. in.

This requires a joist about 8 in. wide and the design must be revised for the extra dead- and live-load, as follows:

Live load = (2.33) (250) = 583. Mom. at center $= \frac{(781)(20)^2(12)}{12}$

Top Slab = (28) (2) = 56. = 312,400 in. lb.

Joist stem $= \left(\frac{8+10}{2}\right)(8) = 72.$ $b = 28$ in. $d = 8.25$ in. $K = \frac{312{,}400}{(28)(8.25)^2} = 164.$

*Formula (103b) may be used, as described on page 6.

Susp. Ceiling $=(2.33)\ (10) \qquad = 23.$
Wood Floor on fill $=(2.33)\ (20) = 47.$
Total load per lin. ft. $\qquad = 781.$

Deficiency in $K = 164 - 117 = 47.$
From Table 16, $p' = 0.012$ and $p = 0.0031$.

The total percentage of tensile reinforcement is:

$$p = 0.0065 + 0.0031 = 0.0096.$$

By formula (103a)* $A_s = (0.0096)\ (28)\ (8.25) = 2.22$ sq. in. = 2 — 1 in. rd. and 1-7/8 in. rd. in bottom.

From Table 1, required width of joist $= 5\frac{1}{2} + 2\frac{3}{16} + \frac{1}{2} = 8\frac{3}{16}$ in. [c]

By formula (105) $A'_s = (0.0031)\,(28)\,(8.25) = 0.72$ sq. in. = 1-3/4 in. rd. and 1-5/8 in. rd. in top.

$$\text{By formula (102a) } v = \frac{(8)\,(10)\,(781)}{(7)\,(8.25)^{(e)}\,(8.25)} = 131 = 0.066f'_c.$$

This requires special anchorage of reinforcement. The design of the joist at the support and of the web reinforcement, follows the same procedure as shown in Problem 2, and will not be repeated here. The flared joist furnished by tapered end-pans will provide the necessary joist width at the support with compressive reinforcement and will reduce the shearing unit stress.

The deflection should be investigated and a camber placed in the forms to equalize the final deflection of the typical and special joists.

$$\text{For the typical joist, } D = (0.0625)\left(\frac{(240)^2}{8.25}\right)(0.00092) = 0.40 \text{ in.}$$

$$\text{For the special joist, } D = (0.0833)\left(\frac{(240)^2}{8.25}\right)(0.00107) = 0.62 \text{ in.}$$

The formwork for the special joists should be cambered 0.22 in. more than that for the typical joists in order to keep a level ceiling.

Notes for Problem 3.

(a) Allowance made for ordinary construction.
(b) Allowance made for 1/4 in. rd. stirrups.
(c) Allowance made for one-half of 3/4 in. bar.

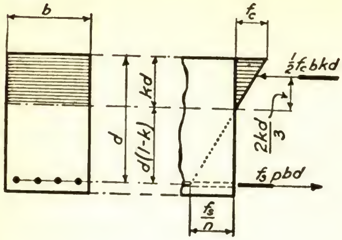
(d) Moment at center increased to $\frac{wl^2}{12}$ on account of special loading.

(e) Width of joist at level of bottom reinforcement = $8\frac{1}{4}$ in.

Combined Bending and Direct Compression.—I have developed no new methods for the design of members subject to bending and direct compression. In general, it is not possible to maintain "balanced reinforcement" in such designs and each diagram must cover a considerable range of stresses both in steel and in the concrete. Diagrams of this same nature. that have been published heretofore, have generally covered too narrow a

*Formula (103b) may be used, as described on page 6.

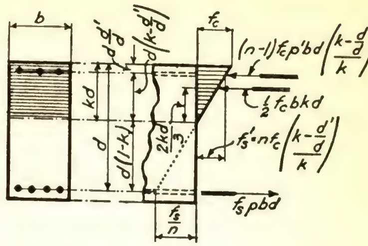
FORMULAS FOR DESIGN VALUES OF p AND K.



$$p = \frac{f_c}{f_s} \cdot \frac{k}{2}$$

$$K = f_c\left(\frac{k}{2} - \frac{k^2}{6}\right)$$

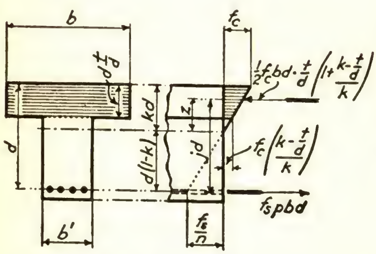
FIG. 3.—RECTANGULAR BEAMS.



$$p = \frac{f_c}{f_s}\left[\frac{k}{2} + \frac{n-1}{k}\left(k - \frac{d'}{d}\right)p'\right]$$

$$K = f_c\left\{\frac{k}{2} - \frac{k^2}{6} + \frac{n-1}{k}p'\left[(1-k)\left(k-\frac{d'}{d}\right) + \left(k-\frac{d'}{d}\right)^2\right]\right\}$$

FIG. 5.—RECTANGULAR BEAMS WITH COMPRESSIVE REINFORCEMENT.

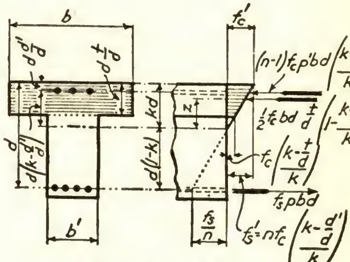


$$z = d\left[k - \frac{t}{d}\left(\frac{3k - 2\frac{t}{d}}{6k - 3\frac{t}{d}}\right)\right]$$

$$p = \frac{f_c}{f_s} \cdot \frac{t}{d}\left(\frac{2k - \frac{t}{d}}{2k}\right)$$

$$K = f_c \cdot \frac{t}{d}\left(\frac{2k - \frac{t}{d}}{2k}\right)\left[1 - \frac{t}{d}\left(\frac{3k - 2\frac{t}{d}}{6k - 3\frac{t}{d}}\right)\right]$$

FIG. 4.—T-BEAMS.



$$z = d\left[k - \frac{t}{d}\left(\frac{3k - 2\frac{t}{d}}{6k - 3\frac{t}{d}}\right)\right]$$

$$p = \frac{f_c}{f_s}\left[\frac{t}{d}\left(\frac{2k - \frac{t}{d}}{2k}\right) + \left(\frac{n-1}{k}\right)\left(k - \frac{d'}{d}\right)p'\right]$$

$$K = f_c\left\{\frac{t}{d}\left(\frac{2k - \frac{t}{d}}{2k}\right)\left[1 - \frac{t}{d}\left(\frac{3k - 2\frac{t}{d}}{6k - 3\frac{t}{d}}\right)\right] + \left(\frac{n-1}{k}\right)p'\left[(1-k)\left(k - \frac{d'}{d}\right) + \left(k - \frac{d'}{d}\right)^2\right]\right\}$$

FIG. 6.—T-BEAMS WITH COMPRESSIVE REINFORCEMENT.

range to be fully adequate for design. Diagrams 20 to 65 cover both circular and rectangular sections for Cases I and II to the full limits of steel ratio permitted by the code, for 2,000, 2,500, 3,000 and 3,750-lb. concrete (and for 5,000-lb. concrete in circular sections) and for ratios of d'/t up to 0.2. All these diagrams have been newly computed for this paper. So far as I know, this is the first time that diagrams for 3,000, 3,750 and 5,000-lb. concrete have been prepared. The essential formulas are given below and Problems 4, 5 and 6 illustrate their application. For the derivation of the formulas and a complete descriptive text the treatment of this subject in "Structural Members and Connections,"† pages 526 to 567, is recommended as probably the most complete of the available discussions.

Special Notations for Bending and Direct Compression.—New symbols, as used in the formulas, diagrams and problems, have the following significance:

e = distance from point of application of N to gravity axis of section.

N = component, normal to section, of all forces acting on it.

L, Q, Z, R_1 = expressions introduced to reduce labor of computations.

p_0 = ratio of total area of symmetrically placed reinforcement to gross area of rectangular sections or to core area of circular columns.

r = distance of c. g. of steel area near compressive face of rectangular section to gravity axis.

r = radius of circular core.

t = total depth of rectangular section.

Design Formulas for Bending and Direct Compression.—For either round or rectangular sections the condition in which the entire cross section is in compression is designated as Case I. The condition in which part of the section is in tension is designated as Case II. Only symmetrical arrangement of reinforcement is covered by the design diagrams and the steel must be equally divided between the two faces in the case of rectangular sections. The usual design formulas follow:

*Round Sections—Case I.**

$$\text{Maximum stress in concrete, } f_c = \frac{NQ}{\pi r^2} \quad \text{.........(106)}$$

The value of Q is taken from Diagrams 20, 21, 22, 23 or 24.

*Round Sections—Case II.**

$$R_1 = \frac{Ne}{\pi r^3} \quad \text{.................................(107)}$$

$$\text{Stress in concrete, } f_c = \frac{R_1}{\dfrac{R_1}{f_c}} \quad \text{..................(108)}$$

* Formulas for circular sections are based on a solution by Mr. C. S. Whitney, Consulting Engineer, Milwaukee, Wisconsin.

† See Hool and Kinne's "Structural Members and Connections."

$$\text{Tensile stress on steel, } f_s = R_1\left(\frac{f_s}{R_1}\right) \qquad \text{(109)}$$

Values of $\dfrac{R_1}{f_c}$ and of $\dfrac{f_s}{R_1}$ are taken from Diagrams 25, 26, 27, 28 or 29.

Problem 4 illustrates the application of these formulas and diagrams to design.

For rectangular sections it must first be determined whether the solution falls under Case I or Case II. Case I formulas and diagrams must be used when the existing value of e/t is **less** than the value of e/t computed from the formula (110).

$$\frac{e}{t} = \frac{1 + 12np_o\left(\dfrac{r}{t}\right)^2}{6(1 + np_o)} \qquad \text{(110)}$$

Case II formulas and diagrams must be used when it is greater.

Rectangular Sections—Case I.

$$\text{Maximum stress in concrete, } f_c = \frac{NZ}{bt} \qquad \text{(111)}$$

The value of Z is taken from Diagrams 30 to 45 for various values of d'/t from 0.05 to 0.2. Problem 5 illustrates the application of this formula and these diagrams to design.

Rectangular Sections—Case II.

$$\text{Stress in concrete, } f_c = \frac{Ne}{bt^2L} \qquad \text{(112)}$$

$$\text{Tensile stress in steel, } f_s = nf_c\left(\frac{d}{kt} - 1\right) \qquad \text{(113)}$$

The value of L is taken from Diagrams 50, 55, 60 or 65 after the value of k has first been determined from Diagrams 46 to 49, 51 to 54, 56 to 59 or 61 to 64. Problem 6 illustrates the application of these formulas and diagrams to design.

The formulas for Q, Z and L may be found in the text referred to above as well as a full statement of the steps to be taken in design and formulas for the general case of non-symmetrical reinforcement for which complete design diagrams are not available.

PROBLEM 4.

A spiral column carrying 400,000 lb. of direct load including its own weight, is also subject to 4,800,000 in. lb. bending moment. Design the column in accordance with the 1928 Joint Code with 3,000-lb. concrete, and determine the principal stresses.

Solution: Assume a 36-in. diameter column with a 32-in. round core and $p = 0.04$.

$$e = \frac{4{,}800{,}000}{400{,}000} = 12 \text{ in.} \qquad \frac{r}{e} = \frac{16}{12} = 1.33$$

This is obviously Case II, since e is so large.
From Diagram 27, upper portion: $R_1/f_c = 0.296$.
From Diagram 27, lower portion: $f_s/R_1 = 18.0$

By formula (107) $R_1 = \dfrac{M}{\pi r^3} = \dfrac{4,800,000}{(3.14)(16)^3} = 373$

By formula (108) $f_c = \dfrac{373}{0.296} = 1,260$ lb. per sq. in.

(1,530 lb. per sq. in. is permitted by code *without wind allowance.*)
By formula (109) $f_s = (18.0)(373) = 6,720$ lb. per sq. in.
The core area by Table 105 is 804.2 sq. in.
The steel area will be:

$A_s = (0.4)(804.2) = 32.2$ sq. in. $= 21$-1¼ in. sq. bars spaced uniformly inside the spiral.

The spiral will be determined by the design of this same column for the *direct load* only by the method shown in Problem 9.

PROBLEM 5.

A rectangular tied column, whose lesser dimension is limited to 20 in. carries a direct load of 800,000 lb. including its own weight and is also subject to a bending moment of 2,400,000 in. lb. acting in a plane perpendicular to the longer dimension of the column. Design the column in accordance with the 1928 Joint Code with 3,000-lb. concrete, and determine the principal stresses.

Solution: From the limit given in the problem, $t = 20$ in. Assume $p = 0.02$ and $d' = 0.15t$, making $r = 0.35t$.

$e = \dfrac{2,400,000}{800,000} = 3$ in. $\quad \dfrac{e}{t} = \dfrac{3}{20} = 0.15$

By formula (110) $\dfrac{e}{t} = \dfrac{1 + (120)(.02)(.35)^2}{6 + (60)(.02)} = .18$

This computed value of $\dfrac{e}{t}$ is greater than the existing value by the conditions of the problem and the diagram for Case I (Compression over entire section) apply.

From Diagram 40, $Z = 1.52$.

By formula (111) $b = \dfrac{(800,000)(1.52)}{(1,350)(20)} = 45$ in.

The concrete stress is 1,350 lb. per sq. in. as used in solving formula (111) above. The steel stress will be low since no tension occurs.

A column 45 in. wide by 20 in. deep is required for bending but it must be made 50 in. wide to carry the direct load.

The steel area will be:

$A_s = (0.02)\ (50)\ (20) = 20$ sq. in. $= 20$-1 in. sq. bars placed symmetrically with ten 1 in. sq. bars at each face.

The value of d' will be $2 + \frac{1}{2} = 2.50$ in. and $d' = 0.125t$ which checks the value assumed.

The ties will be ¼-in. rd. at 12 in. o.c. and provided with legs through the center of the column in each direction in addition to the outer tie enclosing all the bars.

PROBLEM 6.

A square tied column carries a direct load of 200,000 lb. including its own weight and is also subject to a bending moment of 4,000,000 in.-lb. Design the column in accordance with the Joint Code with 3,000-lb. concrete, and determine the principal stresses.

Solution: Assume a column 26 in. by 26 in. and assume $p = .03$. By this assumption, $t = 26$ in. and $d' = 2.6$ in. $= 0.1t$ allowing 2-in. fireproofing.

$$e = \frac{4{,}000{,}000}{200{,}000} = 20 \text{ in.} \qquad \frac{e}{t} = \frac{20}{26} = 0.77$$

By formula (110) $$\frac{e}{t} = \frac{1 + (120)\,(.03)\,(.40)^2}{6 + (60)\,(.03)} = 0.20$$

This is Case II, since 0.20 is less than 0.77

From Diagram 57, with $\dfrac{e}{t} = 0.77$ and $p = .03$; $k = 0.49$

From Diagram 60, Part I, with $p = .03$ and $k = 0.49$; $L = 0.18$

By formula (112) $$f_c = \frac{4{,}000{,}000}{(26)\,(26)^2(0.18)} = 1{,}263 \text{ lb. per sq. in.}$$

A slightly smaller value of p may be determined by trial to give a higher concrete stress if it is desired to use the full 1,350 lb. per sq. in. permitted by code *when wind load is used.*

The steel area will be:

$A_s = (.03)\,(26)^2 = 20.3$ sq. in. $= 13$-1¼ in. sq. bars

Since the concrete stress is low we may use 12-1¼ in. sq. bars and secure a symmetrical arrangement of the bars.

By formula (113) the tensile stress in the steel will be:

$$f_s = (10)\,(1{,}263) \left[\frac{23.4}{(0.49)\,(26)} - 1 \right] = 10{,}550 \text{ lb. per sq. in.}$$

Web Reinforcement—General.—The last step in the calculations for each of the four types of beams calls for the design of the web reinforcement. In this paper all formulas for the design of web reinforcement are

BASIS OF FORMULAS FOR WEB REINFORCEMENT.

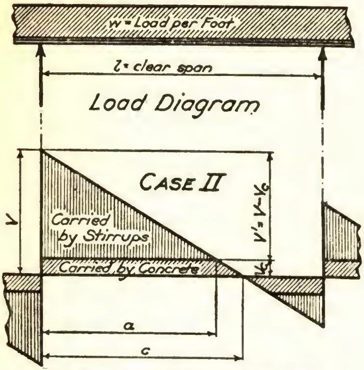


FIG. 7.—SHEAR DIAGRAM FOR UNIFORM LOAD ONLY.

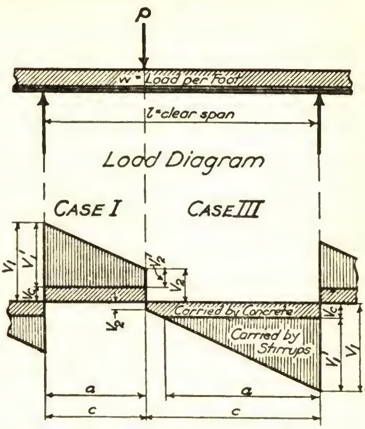


FIG. 9.—SHEAR DIAGRAMS WITH CONCENTRATED AND UNIFORM LOADS.

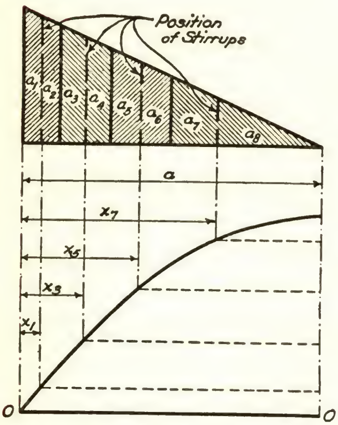


FIG. 8.—STIRRUP SPACING FOR CASES II AND III.

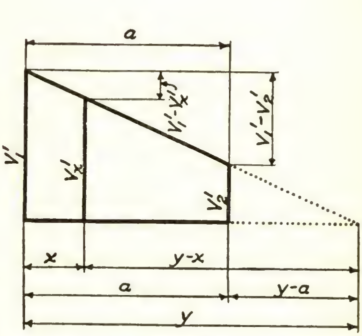


FIG. 10.—STIRRUP SPACING FOR CASE I (ENLARGED FROM FIG. 9).

based on the vertical shears in the usual manner. The web reinforcement computed from them will be adequate in amount and properly arranged to resist the diagonal tensile stresses which occur in the web. In Fig. 7 and Fig. 9 are shown the three shapes of areas that may occur in the shear diagram between the axis and the shear curve. Each area represents the total shear to be resisted by the web concrete and reinforcement. In Fig. 7, with uniform load only on the beam, these areas are shown to be triangles. This is designated as Case II and is a special form of the general trapezoidal case. In Fig. 9, with both uniform and concentrated loads on the span, these areas are trapezoids. At the left, the minimum shear at the low end of the trapezoid is greater than V_c, the safe resistance of unreinforced web concrete. This is designated as Case I. On the right the minimum shear at the low end is less than V_c. This is designated as Case III. Fig. 8 illustrates the law governing stirrup spacing. Fig. 10 is introduced to aid anyone who wishes to check the formulas in the following treatment but is of no interest otherwise.

In design by the Joint Code, the area under the shear curve is considered as made up of two parts. The first part is the resistance of the concrete itself at the unit shearing stress permitted by Section 306 of the Code. This shear resistance at any section is determined by formula (114).

$$V_c = \frac{7}{8} v_c bd \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (114)$$

In Figs. 7 and 9, this value of the shear on the concrete, V_c is laid off and a line drawn parallel to the axis, defining the area (shown by diagonal cross-hatching) which represents the portion of the total area under the shear curve which may be considered as carried by the concrete without the aid of web reinforcement. This leaves the second part of the area under the shear curve (shown in Figs. 7 and 9 by vertical cross-hatching) as the measure of the shear resistance which must be provided by the web reinforcement.

For vertical stirrups, the total resistance, $\Sigma V'$, with N stirrups, is found by formula (115).

$$\Sigma V' = \frac{7}{8} N A_v f_v d \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (115)$$

For inclined stirrups, the total resistance, $\Sigma V'$, is found by formula (116).

$$\Sigma V' = \frac{7}{8} N A_v f_v d \text{ cosec } \alpha \quad \ldots\ldots\ldots\ldots\ldots\ldots (116)$$

The value of $\Sigma V'$ is the vertically-hatched area under the shear diagram and should be computed taking the shear in pounds and the distance along the axis in inches. From this, the number and size of stirrups may be computed by formulas (115) and (116). Using the stirrup steel stress permitted by the code (16,000 lb. per sq. in.) and transposing, formulas

(115) and (116) reduce to forms convenient for design as given in formulas (117) and (118). For vertical stirrups,

$$NA_v = \frac{\Sigma V'}{14{,}000d} \quad \ldots\ldots (117)$$

For inclined stirrups,

$$NA_v = \frac{\Sigma V' \sin a}{14{,}000d} \quad \ldots\ldots (118)$$

in which A_v = right cross-sectional area of a single stirrup (both legs of a U-stirrup) and a = angle between the stirrup and the horizontal.

Table 67 gives all values of NA_v for 1 to 20 U-shaped stirrups of the usual sizes, for use with formulas (117) and (118). In using Table 67, the maximum permissible size of stirrup for any value of d should be determined from Diagram 66. If larger stirrups are used, the anchorage must be increased or the stress decreased. In any case the area $\Sigma V'$ (the vertically-hatched portion in Fig. 7 or Fig. 9) must be computed, taking the shears in pounds and distances along the beam axis in *inches*. In this computation the distance, a, along the axis, requiring web reinforcement must be determined. In Case I, a is equal to the distance from the face of the support to the load; or if a trapezoid between two loads were involved, a is equal to the base of the particular trapezoid under the shear curve for which stirrups are being designed. For Case II, with a triangular area under the shear curve, the value of a is found from formula (119).

$$a = \left(\frac{V - V_c}{V}\right)(c) \quad \ldots\ldots (119)$$

in which c = the base length of the triangle in inches. For Case III in like manner (see Fig. 9 and Fig. 10) the value of a is given by formula (120).

$$a = \left(\frac{V'_1}{V_1 - V_2}\right)(c) \quad \ldots\ldots (120)$$

Spacing of Stirrups.—Having calculated the number and size of stirrups required as shown above the spacing must be determined. Each stirrup should be so located as to take care of two equal unit trapezoids under the shear curve (see Fig. 8 in which a_1, a_2, a_3, etc., are the equal unit areas, each equal to $\Sigma V' \div 2N$) and should therefore be located at the junction line between these areas, for example between a_1, and a_2, a_3 and a_4, etc. The distances from the high end of the trapezoid under the shear curve to these points are designated x_1, x_3, x_5, etc., and are determined by formulas (121), (122), etc.:

$$\frac{x_1}{a} = \frac{V'_1}{V'_1 - V'_2} - \sqrt{\left(\frac{V'_1}{V'_1 - V'_2}\right)^2 - \left(\frac{V'_1 + V'_2}{V'_1 - V'_2}\right)\left(\frac{1}{2N}\right)} \quad \ldots\ldots (121)$$

$$\frac{x_3}{a}=\frac{V'_1}{V'_1-V'_2}-\sqrt{\left(\frac{V'_1}{V'_1-V'_2}\right)^2-\left(\frac{V'_1+V'_2}{V'_1-V'_2}\right)\left(\frac{3}{2N}\right)} \quad \text{....} (122)$$

and so on, the number in the numerator of the final fraction in each formula corresponding to the subscript of x. Diagrams 69, 70 and 71 give values of $\frac{x_1}{a}$, $\frac{x_3}{a}$, $\frac{x_5}{a}$, etc., for all variations in the ratio of $\frac{V'_2}{V'_1}$ and for 1 to 20 stirrups. Having determined a, by formula (119) or (120) or directly from the shear diagram, the distances to the stirrup locations are taken directly from Diagrams 69, 70 and 71, using 72 as an aid, as explained in the instructions for their use.

With uniform load only on the beam, Case II, the values of V'_2 and of $\frac{V'_2}{V'_1}$ become zero and the values of $\frac{x_1}{a}$, $\frac{x_3}{a}$, etc., appear on the lower line of each section of Diagrams 69, 70 and 71. These same values are arranged in Table 68 with spacing grouped in the usual practical manner, and afford an especially rapid but accurate computation of stirrup spacing for uniform load.

Total Number of Stirrups.—The number of stirrups, N, found from the diagrams is the theoretical minimum number necessary to provide the required resistance, $\Sigma V'$. In the final design account must be taken of the limitations imposed by Section 804 of the Code. If the theoretical spacing is written down as it is read from the diagram a casual inspection will show how many stirrups must be added to comply with the rules governing maximum stirrup spacing. Problem 7 shows the complete solution of a general case, including the consideration of extra stirrups to meet these rules, while Problem 1 shows a solution using Table 68.

More Than 20 Stirrups Required.—For this case the designer will generally resort to a UU-shaped stirrup rather than to use too close a spacing. More than twenty stirrups to a single trapezoid under the shear curve is ordinarily undesirable. With UU-stirrups the values in Table 67 will be doubled. If, however, the designer desires to use more than 20 stirrups he may proceed as indicated in Fig. 11, dividing the original trapezoid into two smaller ones and making two solutions. If the lower trapezoid represents the shear value of 20 U-stirrups, the spacing in the higher of the two trapezoids may be considered as uniform and equal to the value by formula (123).

$$s=\frac{14{,}000\, A_v\, dx}{\Sigma V'} \quad \text{..........................} (123)$$

in which $\Sigma V'$ is the area of the higher of the two trapezoids. The end

spacing will be $\frac{s}{2}$. The division point in Fig. 11 will be at a distance, x, from the high side of the original trapezoid determined by formula (124).

$$\frac{x}{a'} = \frac{V'_1}{V'_1 - V'_2} - \sqrt{\left(\frac{V'_1}{V'_1 - V'_2}\right)^2 - \left(\frac{V'_1 + V'_2}{V'_1 - V'_2}\right)\left(\frac{N' - 20}{N'}\right)} \quad \text{..(124)}$$

Bent-up Bars as Web Reinforcement.—Fig. 12 illustrates the common case of a bar bent up in crossing from the bottom of the beam to the top of the beam. Under the Joint Code such a bar may be considered as effective web reinforcement over the center three-quarters of its sloping portion and to have a value over this distance, a_2, determined from formula (125).

$$V' = 16{,}000 A_v \sin a \quad \text{.......................... (125)}$$

In Fig. 12 the unshaded area represents the portion of $\Sigma V'$ taken by this

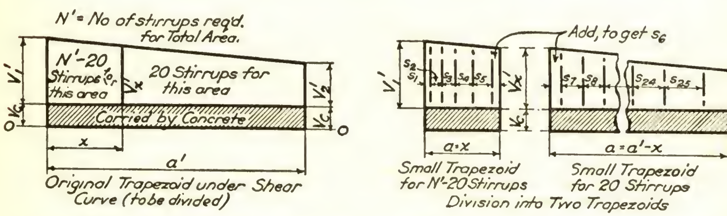


FIG. 11.—METHOD OF APPLYING SPACING DIAGRAMS WHEN MORE THAN TWENTY STIRRUPS ARE USED.

bar, leaving the two trapezoids, 1 and 2, and the triangle, 3, to be reinforced by stirrups. Values of V' for bars bent up at various slopes are given in Diagram 73. Problem 7 shows the complete design of a beam in which such a bent-up bar provides a portion of the web reinforcement at one end.

PROBLEM 7.

For purposes of comparison design the web reinforcement at one end of the beam shown by Figure 13, (a) using vertical stirrups and (b) using bent-up beam bars in conjunction with stirrups. Assume 2,000-lb. concrete in each case.

Solution: The beam is 12 by 36 in. For fire resistive construction $d = 36 - (1\frac{1}{2} + \frac{1}{2} + \frac{5}{8}) = 33.4$ in.

By formula (114) $V_c = (0.02)(2{,}000)(\frac{7}{8})(12)(33.4) = 14{,}000$ lb.

Laying this off from the shear axis as shown in Fig. 13 the value of V'_1 is 15,000 and of $V'_2 = 12{,}150$ and the ratio is:

$$\frac{V'_2}{V'_1} = \frac{12{,}150}{15{,}000} = 0.81$$

This is Case I and the value of a is 76 in. by inspection.

(a) *Design Using Vertical Stirrups.*—The area under the shear curve to be carried by the vertical stirrups, shown vertically-hatched in the diagram is:

$$\Sigma V' = \left(\frac{12{,}150 + 15{,}000}{2}\right)(76) = 1{,}030{,}000 \text{ in. lb.}$$

From Diagram 66 the maximum size of vertical stirrup for $d = 33.4$ and 2,000 lb. concrete is ⅜ in. rd., if plain, or ½ in. rd., if deformed. Use

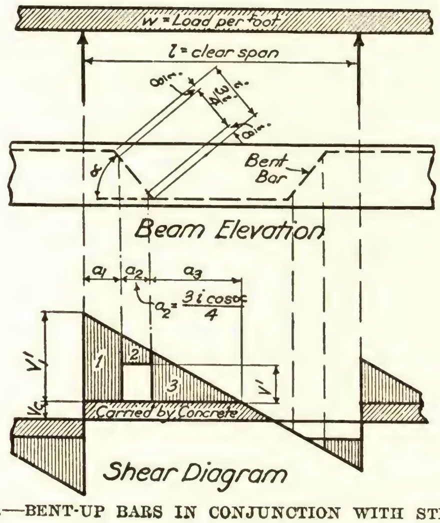


FIG. 12.—BENT-UP BARS IN CONJUNCTION WITH STIRRUPS.

½ in. rd. deformed bar for stirrups. By formula (117)

$$NA_v = \frac{1{,}030{,}000}{(14{,}000)(33.4)} = 2.21 \text{ sq. in.}$$

From Table 67, 6-½ in. rd. U-stirrups provide 2.36 sq. in. Enter Diagram 69, using the second section from the bottom (for 6 stirrups); the distance from the face of the support for $V'_2 \div V'_1 = 0.81$ are:

1st stirrup	$= 0.08a = (0.08)(76) = 6$ in.	Stirrup spacing 6 in.
2nd stirrup	$= .24a = (0.24)(76) = 18$ in.	Stirrup spacing 12 in.
3rd stirrup	$= .39a = (0.39)(76) = 30$ in.	Stirrup spacing 12 in.
4th stirrup	$= .56a = (0.56)(76) = 43$ in.	Stirrup spacing 13 in.

5th stirrup = $.74a = (0.74)(76) = 56$ in. Stirrup spacing 13 in.
6th stirrup = $.91a = (0.91)(76) = 69$ in. Stirrup spacing 13 in.

Since $v = \dfrac{(8)(29{,}000)}{(7)(12)(33.4)} = 83$ lb. per sq. in. $(= .042 f'_c)$ the maximum spacing by section 804 of the code is $(\frac{3}{4})(33.4) = 25$ in. and the design is satisfactory, calling for 6-½ in. rd. U-stirrups spaced 6 in., 2 at 12 in., 3 at 13 in., starting at the face of the support at each end of the span.

(b) *Design Using Bars Bent-up in Single Plane.*—By section 805c of

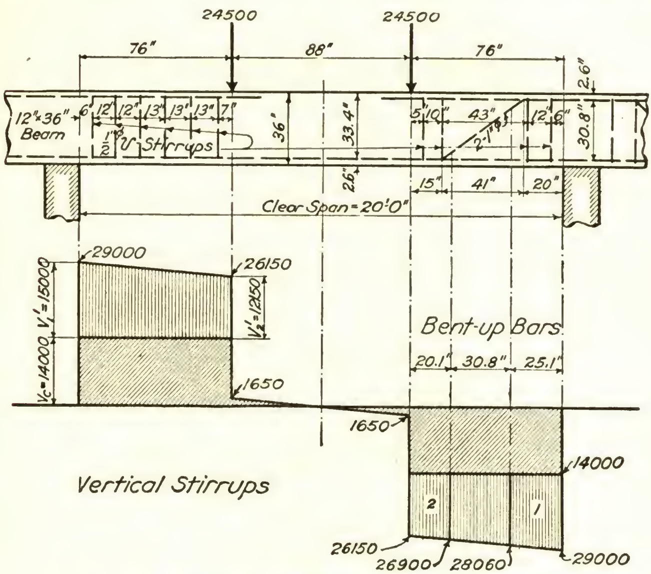


FIG. 13.—STIRRUPS VS. BENT-UP BARS (PROBLEM 7).

the code, only the center three-quarters of the sloping portion may be considered effective as web reinforcement.

It will generally prove wasteful to attempt to bend the bar so that the effective zone will cover the entire 76 in. that requires web reinforcement. The points of bend would have to lie $(1/6)(76) = 12.7$ in. inside the face of the support and beyond the load point, so that the bar would be of very little value as tensile reinforcement at sections of maximum negative and positive moment. It will be better to add stirrups at either end of the trapezoid and make the upper point of bend 20 in. outside the face of the support and the lower point of bend 15 in. inside the load point (see Fig. 13) where the moments are such that this steel may be bent down

without requiring extra tensile reinforcement. The length of the "run" of the bar (Diagram 73) will be 76 — (15 + 20) = 41 in. The "rise" will be 36 — [(2) (2.6)] = 30.8. Ratio of "run" to "rise" $= \dfrac{41}{30.8} = 1.33$. From Diagram 73, lower part, the value of 1 — 1 in. rd. bar at this ratio is 7,500 lb. and 2 — 1 in. rd. bars will be required to carry $V'_1 = 15{,}000$ lb.

The zone, a_2, in which these bars are effective will end at (⅛) (41) = 5.1 in., from the points of bending of the bar and will be 41 — [(2) (5.1)] = 30.8 in. wide located as shown in Fig. 13. The 25.1 in. at the support will require 2-½ in. rd. U's while the 20.1 in. next to the load point will require 2-½ in. rd. U's, as is readily determined by the stirrup design already made. The stirrup spacing from the face of the support may be taken as 6 in., 12 in., 38 in., 13 in. from the previous stirrup design.

A computation of the stirrups required by the small trapezoid 2 will give a check on the results as follows:

$$\Sigma V' = \left(\frac{12{,}900 + 12{,}150}{2}\right)(20.1) = 252{,}000 \text{ in. lb.}$$

$$NA_v = \frac{252{,}000}{(14{,}000)\,(33.4)} = 0.54 \text{ sq. in.} = 2\text{-}\tfrac{1}{2} \text{ in. rd.}$$

$$\frac{V'_2}{V'_1} = \frac{12{,}150}{12{,}900} = 0.94 \qquad a = 20.1 \text{ in.}$$

From Diagram 69 the distance from edge of zone covered by bent-up bars to first stirrup = (0.25) (20.1) = 5 in.
To second stirrup = (0.75) (20.1) = 15 in.

Revised spacing of stirrups will be 6 in., 12 in., 43 in., 10 in.

Two-way Slabs Supported on Beams.—Diagram 74 gives the load distribution for design strips (commonly taken as 12 in. wide) in each direction in ordinary slabs supported on beams on all four edges. The design of the unit strips within the middle half of the clear span in each direction employs the same moment coefficients as are used for beams under the same general conditions of loading, support and restraint. In the outer quarters the reinforcement is permitted to be reduced to one-half of that required in the parallel middle strips. The supporting beam must be designed to carry in addition to its own weight and superimposed live load a uniform load throughout its length equal to the load per foot brought to it by the middle strips on either side. No reduction in live load is permitted on beams supporting two-way slabs, even though such reduction may be used under the code for beams supporting one-way slabs. The design of the strips is the same as the design of rectangular beams.

Quantities of Concrete, Formwork and Reinforcement.—In the design of beams and T-beams architectural considerations, such as unobstructed head room, arrangement of beams on ceiling, etc., are so important that

the range in beam sizes and proportions, even in buildings of the same span and floor load, is considerable. I have not attempted, therefore, to give any formulas or diagrams for quantities of materials required in the beam-and-slab types of structure. Part II of this paper, however, gives tables of quantities from actual designs for quite a wide range of panel sizes, live loads and concrete strengths. In the case of columns, likewise, there is ordinarily some architectural limitation on size that introduces wide diversity in designs for the same vertical load, and there is very commonly some bending moment present to modify the final design. It is therefore difficult in this case also to prepare any formulas or diagrams for quantities of materials that would be generally applicable. Where members can be standardized, as is the case with flat slab floors and with spread footings, it is not difficult to prepare diagrams giving quite closely the volume of concrete, area of formwork and weight of reinforcing steel required for typical design conditions.

For flat slab floors each *design* diagram is followed on the facing page by a *quantity* diagram for the same conditions. The quantities of concrete, formwork and reinforcement required for a design taken from Diagram 77, for example, may be read directly from Diagram 78. For square spread footings the same arrangement has been followed. The quantities for a design selected from Diagram 112, for example, may be taken directly from Diagram 113 on the facing page. In the case of footings this offers the additional advantage that the *excavation* required by the contractor may be taken from the design diagram (making proper allowances for the space occupied by the formwork) in the same operation without turning a page.

In using quantity diagrams allowance must always be made for extra concrete if a slab is made 8¼ inches thick where the design diagram calls for 8.2 inches, for example. The diagrams are right, but the actual design may vary slightly from the theoretical and a corresponding allowance must be made by the estimator. In the same way some allowance should always be made for extra steel in cases where the steel area from the diagram does not divide evenly into bars. The diagram may call for 6.8, ½-in. round bars, but the design will call for seven, and the estimator must add for this contingency. It is impossible to make such allowances in the diagrams since one designer may use ½-in. round bars while another may use ⅝-in. round bars, and the allowance would be different.

These quantity diagrams are useful to the designer as well as to the estimator. It enables the designer to study the relative economy for different concretes, varying spans and soils of unequal bearing power, with a great saving in time and labor. In using quantity diagrams for comparative purposes in design it is not necessary to make any allowances, as mentioned in the last paragraph, since these allowances will balance very closely and will not affect the comparisons materially.

Flat Slab of Standardized Proportions.—The Joint Code properly permits a wide latitude in the proportions of flat slab floors, to permit of columns without capitals, or without drops, or with unusually large or

small capitals, such as frequently are required. For the usual run of factory buildings, however, a standardized design is entirely acceptable and will save much labor. I have used a column capital diameter equal to 0.225 l in Diagrams 79, 83, etc., while in Diagrams 77, 81, etc., the capital size varies from 0.225 l to 0.25 l according to standard metal column form used. Diagrams 79, 83, etc., may be used with wood column capital forms where no standards govern but only Diagrams 77, 81, etc., should be used for circular capitals formed in metal molds in the usual way. The side of the square dropped panel is taken as 0.35 l in all cases. The depth of the dropped panel below the slab is taken as one-half the slab thickness in all cases. The following moments are taken which lie within the values given in table in Section 1003 of the code:

Two-way system	*Four-way system*
$-M_c = 0.47\,M_o;\ +M_c = 0.21\,M_o$	$-M_c = 0.51\,M_o;\ +M_c = 0.20\,M_o$
$-M_m = 0.16\,M_o;\ +M_m = 0.16\,M_o$	$-M_m = 0.09\,M_o;\ +M_m = 0.20\,M_o$

This distribution of the total bending moment gives somewhat different values for the square four-way panel than those stated in Section 1004c of the code, which is merely one of several distributions permitted. The distribution used in the diagrams and tables of this paper results in a simpler design for the standardized proportions. Designs made in accordance with this office standard may be taken direct from Diagrams 77, 79, etc., in accordance with the instructions under Table 75, for all cases of square interior flat slab floor panels surrounded by other panels of approximately the same size. Problem 8 shows a complete design for such a panel for both two-way and four-way systems. The length of bars in the various bands must be determined from the provisions of Sections 1007 to 1010 of the Joint Code.

Where exterior panels in flat slab floors are of the same size and shape as the adjoining interior panels and have regular column capitals, the column strip or direct band lying partly in the interior panel and partly in the exterior panel will be the same as for an interior panel by Table 75. The middle strip of a two-way system parallel to the wall will be the same as for an interior panel by Table 75. The column strip or direct band along the wall will be (proportional to its width) the same as a similar interior strip or band except as affected by the provisions of Section 1012 of the code. The top band across the direct band, and extending from an interior to an exterior panel, and also extending between two exterior panels will be the same as for an interior panel by Table 75. The remaining design strips or bands will take the reinforcement called for by Table 76. The slab thickness, drop thickness, etc., will be governed by Table 75 and Diagrams 77, 79, 81, etc., to 91.

For rectangular, irregular or special panels, the tables and diagrams are not applicable and the usual complete design process must be resorted to. Even for such cases, however, the diagrams afford an excellent basis

for judgment in assuming slab thicknesses, etc., or in making rough estimates.

PROBLEM 8.

Design the typical square interior and exterior panels of a flat slab floor (*a*) with four-way reinforcement and (*b*) with two-way reinforcement, assuming 2,000-lb. concrete. The wall columns have a half regular column capital. Use a live load of 200 lb. per sq. ft. and the Joint Code, with standard steel capital forms. All panels are 21 ft. by 21 ft. c. to c. of columns.

(a) *Design with Four-way Reinforcement.*—Solution: Diagram 77 in conjunction with Tables 75 and 76 give the design as follows:

From Diagram 77 for a 21 ft. square panel:

Side of square dropped panel = 7 ft. 4 in.
Slab thickness for 200-lb. LL = 8⅜ in.
The column capital will be 5 ft. in diameter.
Basic steel area for 200-lb. LL = 1.58 sq. in.

From Table 75:

Dropped panel thickness = (½) (8⅜) = 4¼ in.
This makes the dropped panel, 7 ft. 4 in. by 7 ft. 4 in. by 4¼ in.
Each top band, A_s = 1.58 sq. in. = 8-½ in. rd. bars.

For the typical interior panel:

Diagonal band—bent bars, A_s = (0.67) (1.58) = 1.05 sq. in. = 6-½ in. rd.

Diagonal band—straight bars, A_s = 1.58 sq. in. = 8-½ in. rd.
Direct band—bent bars, A_s = 1.58 sq. in. = 8-½ in. rd.
Direct band—straight bars,
A_s = (1.22) (1.58) = 1.93 sq. in. = 10-½ in. rd.

From Table 76, for the typical exterior panel:

Top band at and perpendicular to wall,
A_s = (0.625) (1.58) = 0.99 sq. in. = 5-½ in. rd.
Diagonal band—bent bars,
A_s = (1.13) (1.58) = 1.79 sq. in. = 9-½ in. rd.
Diagonal band—straight bars,
A_s = (0.75) (1.58) = 1.18 sq. in. = 6-½ in. rd.
Diagonal band—top bars over exterior col. head,
A_s = (0.22) (1.58) = 0.35 sq. in. = 2-½ in. rd.
Direct band, perpendicular to wall,—bent bars,
A_s = (1.67) (1.58) = 2.64 sq. in. = 14-½ in. rd.
Direct band, perpendicular to wall,—straight bars,
A_s = (1.11) (1.58) = 1.75 sq. in. = 9-½ in. rd.

The direct band, parallel to the wall, lying partly in the interior and partly in the exterior panel will be the same as the direct band of a typical interior panel.

The direct band (or half band, in most cases) lying along the wall will be designed in accordance with Section 1011 of the code.

(b) *Design With Two-way Reinforcement.*—The slab thickness, the dimensions of the square dropped panel and the column capital will be the same as for the four-way design above. The basic steel area will be 1.58 sq. in. also.

For the typical interior panel, from Table 75:

Middle strip—bent bars,

$A_s = (0.89)(1.58) = 1.41$ sq. in. = 7-½ in. rd.

Middle strip—straight bars,

$A_s = 1.58$ sq. in. = 8-½ in. rd.

Column strip—bent bars,

$A_s = (1.56)(1.58) = 2.46$ sq. in. = 13-½ in. rd.

Column strip—straight bars,

$A_s = (0.78)(1.58) = 1.23$ sq. in. = 6-½ in. rd.

Column strip—added over col. head,

$A_s = (0.36)(1.58) = 0.57$ sq. in. = 3-½ in. rd.

For the typical exterior panel, from Table 76:

Middle strip, perpendicular to wall, bent bars,

$A_s = (1.11)(1.58) = 1.76$ sq. in. = 9-½ in. rd.

Middle strip, perpendicular to wall, straight bars,

$A_s = (1.12)(1.58) = 1.77$ sq. in. = 9-½ in. rd.

Column strip, perpendicular to wall, bent bars,

$A_s = (1.95)(1.58) = 3.08$ sq. in. = 16-½ in. rd.

Column strip, perpendicular to wall, straight bars,

$A_s = (0.98)(1.58) = 1.54$ sq. in. = 8-½ in. rd.

Column strip, perpendicular to wall, bars in top over exterior column head,

$A_s = (1.18)(1.58) = 1.86$ sq. in. = 9-½ in. rd.

The intermediate strip parallel to the wall will have the same bars as an intermediate strip in an interior panel.

The column strip lying partly in the interior and partly in the exterior panel will be the same as the column strip of an interior panel.

The column strip lying along the wall will be designed in accordance with Section 1011 of the code.

(c) *Length of Bars.*—For either four-way or two-way reinforcement the points of bending, etc., and the points of termination of bars will be as given in Sections 1007 to 1010 of the code.

Long Columns.—The upper portion of Diagram 93 gives the ratio of the radius of gyration of a circular core with not over 6 per cent vertical reinforcement to the core diameter. The lower portion of Diagram 93 gives the proportionate load-carrying capacity of columns in which h/R exceeds 40 or 50 in accordance with formula (26) or (26a) of the code. The upper portion of Diagram 93 is based on the approximation that the effective diameter for the ring of longitudinal bars will be 0.9 of the core diameter. This will not apply to columns having the bars arranged in two rings or

to very small or very large columns. Such cases require that the value of R be computed.

Spiral Columns.—In spiral column design, having assumed a core diameter, the area of the spiral core is taken from Table 105 and the weight of the corresponding column is added to the applied load to secure the total design load on the column. Table 106 gives the volumes of square and round column shafts and column capitals. Compute the value of P/A and enter Diagram 94, or 95 to 98, according to the strength of concrete used in the design. Lay the edge of a triangle from the value of P/A on the left scale to the core diameter on the right scale and read off the spiral rod and pitch on the left center scale and the number and size of verticals on the right center scale, completing the design in a single operation. The percentage of spiral in Diagrams 94 to 98 is always one-fourth of the percentage of verticals as required in the code. Problem 9 shows the complete design of a spiral column by this brief method. The provisions of Section 1103 of the Joint Code regarding minimum requirements must be met.

PROBLEM 9.

Design an axially loaded reinforced-concrete column for a load of 1,100,000 lb., including assumed column weight, using 3,000-lb. concrete and the spiral type of column reinforcement, in accordance with the Joint Code. The unsupported length is 11 feet.

Solution: Assume 36-in. round column with a 32-in. diameter core section. The core area from Table 105 is 804.2 sq. in.

$$P/A = \frac{1,100,000}{804.2} = 1,368 \text{ lb. per sq. in.}$$

From Diagram 96, using a straight edge set to 1,368 on the left scale and 32 on the right scale, we read on the center scales

Vertical bars $= 18 - 1\frac{1}{4}$ in. sq.
Spiral $= \frac{1}{2}$ in. rd. at $2\frac{3}{4}$ in. pitch.

The ratio of unsupported length to core diameter $\left(\frac{132}{36} = 3.7\right)$ is so small as not to require consideration in connection with Section 1108 of the code.

Tied Columns.—Diagram 100 is the usual design diagram for tied columns, using the Joint Code. In the usual design procedure, a column size is assumed, the area of which may be taken from Table 107 for ordinary round columns or cylindrical piers, the weight of the corresponding column is added to the applied load and from the value of P/A the proper percentage of vertical reinforcement is read from Diagram 100 for the concrete strength used in the design. The area of vertical rods required will be p times the column area and the corresponding bars may be computed from Table 2 with code limitations observed. The design is completed by the selection of ties to meet the code requirements, Section 1104. Problem 10 shows the complete design of a tied column.

PROBLEM 10.

Design an axially loaded reinforced-concrete column for a load of 300,000 lb., including assumed column weight, using 2,000-lb. concrete and tied longitudinal bars, in accordance with the Joint Code. The unsupported length is 9 feet.

Solution: Assume a column 24-in. square. The area will be (24) (24) $= 576$ sq. in. and the unit load will be $P/A = \dfrac{300,000}{576} = 520$ lb. per sq. in.

From Diagram 100 for $P/A = 520$ and 2,000-lb. concrete the value of p is 0.011.

$A_s = (0.011)(576) = 6.34$ sq. in. $= 8 - 1$ in. rd. bars.

From Section 1104 of the code, the ties will be ¼ in. rd. spaced 12 in. o. c. and so arranged as to provide ties in two directions for the four bars at the middle of the sides of the column as well as for the four bars at the corners. The ties will be bent so as to keep the longitudinal bars 2 in. in the clear from the column surface at all points.

Composite and Combination Columns.—The design of composite and combination columns in accordance with Sections 1106 and 1107 of the code would require four additional diagrams. Space limitations of this paper indicate that diagrams so infrequently used should be omitted. The plotting data for these diagrams are as follows:

Value of p	Composite Columns								Combination Columns	
	Unit Stress on Reinforced Concrete Section with 1% Spiral Reinforcement and $f'c$ =					Unit Stress on Metal Core under Construction Loads			$\frac{A_s}{A}$	Increase in Stress
	2000	2500	3000	3750	5000	h/R	Structural Steel	Cast Iron		
0.02.......	640	763	886	1067	1375	60	15000	8400	0.1	0.090
0.025......	675	798	920	1100	1437	80	13280	7200	0.2	0.040
0.03.......	710	832	954	1133	1500	100	11580	6000	0.3	0.023
0.035......	745	866	987	1166	1562	120	10000	4800	0.4	0.015
0.04.......	780	900	1021	1200	1625	140	8620	3600	0.5	0.010

Spread Footings.—The Joint Code properly leaves the designer considerable latitude in the design of concrete footings resting directly on the soil. A standardization of proportions, putting all horizontal dimensions in terms of b, the dimension of the side of a square footing, and all vertical dimensions in terms of d, the effective depth, greatly simplifies the formulas and expedites the design. Fig. 16 shows the standard proportions adopted for flat-top footings and Fig. 17 those for sloping-top footings. With these constant ratios, the various design requirements can all be expressed in terms of w, the soil load (neglecting the footing weight which is carried

directly to the soil without shear or moment, but which must be used in computing the area of the footing), b and d. The bending moment at a section in the plane of the face of the pier is found by formula (126).

$$M = 0.6Pb \qquad (126)$$

in which

M is in inch-pounds
P is the load in pounds at the top of the footing
b is the side of the base of the square footing in feet.

For *flat-top* square footings the value of d/b is found by solving formula (127) (answer read directly from Diagram 108).

$$\frac{w}{v_c} = \frac{126\left(\frac{d}{b}\right)\left(8\frac{d}{b}+1\right)}{1-\left(2\frac{d}{b}+.25\right)^2} \qquad (127)$$

For *sloping-top* square footings the value of d/b is found by solving formula (128) (answer read directly from Diagram 108).

$$\frac{w}{v_c} = \frac{270\left(\frac{d}{b}\right)\left(8\frac{d}{b}+1\right)\left(.491-\frac{d}{b}\right)}{1-\left(2\frac{d}{b}+.25\right)^2} \qquad (128)$$

In these formulas v_c may be taken at $0.02f'_c$ or $0.03f'_c$ depending on the anchorage of the footing reinforcement. Diagrams 113, 114, etc., which follow are all based on $v_c = 0.03f'_c$ with all footing bars hooked, but Diagram 108 is perfectly general and applies to any value of v_c.

When using the standard proportions of Fig. 16 or Fig. 17 it is not necessary to compute the bending moment, since the proportions are selected to give concrete stresses at or just under the permitted values. The area of steel in *each* of two directions may be computed directly from formula (129).

$$A_s = \frac{0.05P}{17{,}500\left(\frac{d}{b}\right)} \qquad (129)$$

The value of d/b is known from Diagram 108. To complete the design it is only necessary to select the number and size of bars to make up the required A_s, safeguarding the bond stress by not exceeding the size of bar indicated by Diagram 109. If the bar size, D, is taken as not larger than the value by formula (130) the bond stress will be satisfactory.

$$D = 0.0000427\, bu \qquad (130)$$

For deformed bars with hooked ends as specified in Section 903 of the code formula (130) reduces to $D = 0.0064b$ for 2,000-lb. concrete, and to $D = 0.0096b$ for 3,000-lb. concrete. Diagram 109 gives the maximum allowable bar size for any given value of b by formula (130). These formulas

and the values from Diagram 109 apply to both flat-top and sloping-top footings. In flat-top footings the bond unit stress decreases rapidly at sections away from the face of the pier. In sloping-top footings of the proportions used here, the bond unit stress remains almost constant to a point on and near the top of the slope and then decreases to the edge. Hooked bars are required.

In making a design for a fixed soil pressure which includes the weight of the footing it is necessary to determine the weight per sq. ft. of the footing. For *flat-top* footings this is given by formula (131).

$$w' = 12.5d + 50 \quad \ldots\ldots\ldots\ldots\ldots\ldots (131)$$

For *sloping-top* footings it is given by formula (132).

$$w' = 7.5d + 50 \quad \ldots\ldots\ldots\ldots\ldots\ldots (132)$$

In (131) and (132) the term 50 is the weight of the four inches of concrete beneath the centroid of the reinforcing steel.

The volume of concrete in sloping-top footings will be given very closely by formula (133).

$$\text{Volume} = (0.05d + 0.333)\,b^2 \quad \ldots\ldots\ldots\ldots (133)$$

where d is in inches, b is in feet and the *volume* is in cu. ft.

The minimum edge thickness of sloped-top footings is ten inches by the code. If d is less than 24 in. (28 in. total thickness) the weight of footing and volume of concrete in sloped-top footings will be increased. The quantity diagrams include this increase, but formulas (132) and (133) require adjustment for this case.

The depth of a footing of any fixed proportions depends primarily on the superimposed load and is almost independent of the soil pressure used. Diagram 111 gives the depth of footings of the types shown in Fig. 16 and Fig. 17 for 6,000-lb. soil and the depth for any other usual soil pressure will be within a few per cent of these values for the same load. In general other soil pressures will give slightly less depths. 6,000-lb. soil appears to give the maximum depth within the range of soil pressure usually used under spread footings. Selections from Diagram 111 will not need to be checked for diagonal tension or shear, except to determine a possible slight saving in depth.

Diagrams 112, 114 to 126 are design graphs for sloping-top footings for soil pressures of 3,000, 4,000, 5,000 and 6,000-lb. per sq. ft. Each diagram gives values for two strengths of concrete. Similarly, Diagrams 128, 130 to 142 cover the design for flat-top footings for the same soil pressures and concrete strengths. These diagrams *allow accurately* for the footing weights and give the indicated soil pressure when subjected to the axial load shown in the diagrams. Enter the diagrams at the top with the total applied load at the top of the footing (the basement column load plus the assumed weight of the pier) and drop vertically reading off the value of b, of t (the total footing depth) and of A_s in succession. Complete the design by computing the other dimensions of the footings from b and d, in accordance with Fig. 16 or Fig. 17 and by selecting the number and size of bars to make up the required A_s, using bars not larger than

indicated by Diagram 109. Problem 11 shows the complete design of a square footing of the flat-top type for the same basement story column load used in Problem 9. In Problem 12 the design of the top of the pier for the safe transfer of the load without exceeding Joint Code stresses is shown, using Diagram 110 for a rapid solution of formula (28) of the code.

Quantity Diagrams for Footings.—Facing each design diagram, a diagram giving the quantities of concrete, formwork and reinforcing steel for single footings is given. The basis and use of these diagrams is described more fully on page 30, which should be consulted.

PROBLEM 11.

Design a flat-top square spread footing for the column given in Problem 9, using a 3,000-lb. concrete in the footing and designing for a 4,000-lb. per sq. ft. soil pressure.

Solution: Enter Diagram 130 at the top with the load of 1,100,000 lb. Drop vertically and read off the dimension of the side of the square footing at the upper index line for 3,000-lb. concrete as 17 ft. 6 in. Continue vertically down to the middle index line for 3,000-lb. concrete and read off the depth as 30 in. The footing will be 17 ft. 6 in. by 17 ft. 6 in. by 2 ft. 6 in.

Continue vertically downward to the lower index line for 3,000-lb. concrete and read off the steel area as 26.0 sq. in. in each of two directions. From Diagram 109 the maximum bar size for a 17 ft. 6 in. footing using 3,000-lb. concrete is not limited. Use 21 — 1⅛ in. sq. bars in each direction. The length of each bar including the hooks at each end will be:

$$(17 \text{ ft. } 6 \text{ in.}) + [(20)(1\tfrac{1}{8})] = 19 \text{ ft. } 5 \text{ in. as a minimum.}$$

From Fig. 17 the side of the square pier will be

$$(0.25)(17 \text{ ft. } 6 \text{ in.}) = 4 \text{ ft. } 4\tfrac{1}{2} \text{ in.}$$

The height of the pier, for a 36-in. column as designed in Problem 10, will be:

$$(4 \text{ ft. } 4\tfrac{1}{2} \text{ in.}) - (3 \text{ ft. } 0 \text{ in.}) = 1 \text{ ft. } 4\tfrac{1}{2} \text{ in. (minimum).}$$

PROBLEM 12.

Design the top of the pier in Problem 11 for the transfer of the load from the column of Problem 10 to the pier, in accordance with the Joint Code.

Solution: Section 1205 of the code applies:

A = Area of top of pier = (52.5) (52.5) = 2,756 sq. in.

A' = Area of 32-in. round column = 804 sq. in.

$$A/A' = \frac{2756}{804} = 3.43$$

$$r_a \text{ at the base of the column} = \frac{1{,}100{,}000}{804} = 1368.$$

Enter Diagram 110, upper portion, with $A/A' = 3.43$ and move across to $r_a = 1{,}368$. 3,750-lb. concrete is required, without spiral reinforcement, in the pier.

TABLE 1.—MINIMUM BEAM WIDTHS (IN INCHES)

Size of Bars	Number of Bars in Single Layer of Reinforcing							Add for each Added Bar
	2	3	4	5	6	7	8	
A. ORDINARY CONSTRUCTION—END OF BARS NOT SPECIALLY ANCHORED								
½ in. round	4	5½	7	....	....	....	....	1½
½ in. square	4¼	6	7¾	....	....	....	....	1¾
⅝ in. round	4¼	5⅞	7½	9⅛	10¾	....	....	1⅝
¾ in. round	4⅝	6½	8⅜	10¼	12⅛	....	....	1⅞
⅞ in. round	5 1/16	7¼	9 7/16	11⅝	13 13/16	16	18 3/16	2 3/16
1 in. round	5½	8	10½	13	15½	18	20½	2½
1 in. square	6	9	12	15	18	21	24	3
1⅛ in. square	6½	9⅞	13¼	16⅝	20	23⅜	26¾	3⅜
1¼ in. square	7	10¾	14½	18¼	22	25¾	29½	3¾
B. ORDINARY CONSTRUCTION—END OF BARS SPECIALLY ANCHORED								
½ in. round	4	5½	7	....	....	....	....	1½
½ in. square	4¼	6	7¾	....	....	....	....	1¾
⅝ in. round	4¼	5⅞	7½	9⅛	10¾	....	....	1⅝
¾ in. round	4½	6¼	8	9¾	11½	....	....	1¾
⅞ in. round	4¾	6⅝	8½	10⅜	12¼	14⅛	16	1⅞
1 in. round	5	7	9	11	13	15	17	2
1 in. square	5½	8	10½	13	15½	18	20½	2½
1⅛ in. square	5 15/16	8⅞	11 9/16	14⅜	17 3/16	20	22 13/16	2 13/16
1¼ in. square	6⅜	9½	12⅝	15¾	18⅞	22	25⅛	3⅛

INSTRUCTIONS FOR USE.—This table shows the minimum width of beam stem, without stirrups, in which the longitudinal bars are covered by one inch of concrete and are spaced as follows:

For round bars not specially anchored, 2½ bar diameters between centers.

For square bars not specially anchored, 3 times the side dimension between centers.

For round bars with special anchorage, 2 bar diameters between centers.

For square bars with special anchorage, 2½ times the side dimensions between centers.

To the widths shown in the table additions must be made as follows:

1. Add the width of any stirrup legs placed between the longitudinal bars and the sides of the beam.
2. Add 1 in. for fire resistive construction or 2 in. for special exposure or spalling aggregates.
3. Add extra width as required, in case aggregate exceeds ¾ in. in size, to insure a clear space between bars of not less than 1⅓ times the maximum size of the coarse aggregate.

For more bars of any size than covered by the table or for bars of a different size, add the dimension given in the last column for each such extra bar. (See 1928 Joint Code, Sections 504, 506, and 903.)

TABLE 2.—AREAS, PERIMETERS AND WEIGHTS OF PLAIN BARS

Number of Bars		Sizes of Plain Bars										
		¼ in. round	⅜ in. round	½ in. round	½ in. square	⅝ in. round	¾ in. round	⅞ in. round	1 in. round	1 in. square	1⅛ in. square	1¼ in. square
1	A	0.0491	0.1104	0.1963	0.250	0.3068	0.4418	0.6013	0.7854	1.00	1.2656	1.5625
	o	0.785	1.178	1.571	2.000	1.964	2.356	2.749	3.142	4.00	4.500	5.000
	W	0.167	0.375	0.668	0.850	1.043	1.502	2.044	2.670	3.40	4.303	5.313
2	ΣA	0.10	0.22	0.39	0.50	0.61	0.88	1.20	1.57	2.00	2.53	3.12
	Σo	1.57	2.36	3.14	4.00	3.93	4.71	5.50	6.28	8.00	9.00	10.00
	ΣW	0.33	0.75	1.34	1.70	2.09	3.00	4.09	5.34	6.80	8.61	10.63
3	ΣA	0.15	0.33	0.59	0.75	0.92	1.33	1.80	2.36	3.00	3.80	4.69
	Σo	2.36	3.53	4.71	6.00	5.89	7.07	8.25	9.42	12.00	13.50	15.00
	ΣW	0.50	1.12	2.00	2.55	3.13	4.51	6.13	8.01	10.20	12.91	15.94
4	ΣA	0.20	0.44	0.78	1.00	1.23	1.77	2.41	3.14	4.00	5.06	6.25
	Σo	3.14	4.71	6.28	8.00	7.86	9.42	11.00	12.57	16.00	18.00	20.00
	ΣW	0.67	1.50	2.67	3.40	4.17	6.01	8.18	10.68	13.60	17.21	21.25
Area of 12 Bars		0.59	1.32	2.36	3.00	3.68	5.30	7.22	9.41	12.00	15.18	18.75

INSTRUCTIONS FOR USE.—This table is based on the properties of plain round or plain square and may be used in design for areas and perimeters of deformed bars. For weights of deformed bars manufacturers' tables should be consulted.

Cross sectional areas (A or ΣA) of bars are given in sq. in.

Perimeters (o or Σo) are given in inches.

Weights (W or ΣW) are given in lb. per foot of length.

The last line gives values of $12a_s$ for use in the formula:

$$\text{Spacing of bars in slab (in inches)} = \frac{12a_s}{A_s}.$$

in which $12a_s$ = cross sectional area of 12 slab bars of size used.

A_s = cross sectional area per foot width of slab required.

Areas, perimeters and weights of groups of more than four bars may be easily obtained from the table by simple additions or multiplications.

DESIGN VALUES OF p AND K FOR $f_s = 20,000$

TABLE 3.—RECTANGULAR BEAMS. $f_c = 0.40 f'_c$

2000-lb. Concrete	2500-lb. Concrete	3000-lb. Concrete	3750-lb. Concrete
$n = 15$ $f_c = 800$	$n = 12$ $f_c = 1000$	$n = 10$ $f_c = 1200$	$n = 8$ $f_c = 1500$
$p = 0.0075$ $K = 131$	$p = 0.0094$ $K = 164$	$p = 0.0112$ $K = 197$	$p = 0.0140$ $K = 246$

$k = 0.375$ for all of the above cases.

INSTRUCTIONS FOR USE.—The value of f_c is 0.4 f'_c and is the compressive stress to be used in flexure calculations, except for the special case covered by Table 4. (See also general note under Table 5.)

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 4.—RECTANGULAR BEAMS. $f_c = 0.45f'_c$

2000-lb. Concrete	2500-lb. Concrete	3000-lb. Concrete	3750-lb. Concrete
$n = 15$ $f_c = 900$	$n = 12$ $f_c = 1125$	$n = 10$ $f_c = 1350$	$n = 8$ $f_c = 1688$
$p = 0.0091$ $K = 157$	$p = 0.0113$ $K = 196$	$p = 0.0136$ $K = 235$	$p = 0.0170$ $K = 294$

$k = 0.403$ for all of the above cases.

INSTRUCTIONS FOR USE.—The value of f_c is $0.45\,f'_c$ and its use is limited to designs for flexure adjacent to supports of continuous or fixed beams or of rigid frames. (See also general note under Table 5.)

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 5.—T-BEAMS. $f_c = 0.40\,f'_c$

$\frac{t}{d}$	2000-lb. Concrete		2500-lb. Concrete		3000-lb. Concrete		3750-lb. Concrete	
	$n = 15$, $f_c = 800$		$n = 12$, $f_c = 1000$		$n = 10$, $f_c = 1200$		$n = 8$, $f_c = 1500$	
	p	K	p	K	p	K	p	K
0.04	0.0015	30	0.0019	37	0.0023	45	0.0028	56
0.06	0.0022	43	0.0028	54	0.0033	64	0.0041	80
0.08	0.0029	55	0.0036	69	0.0043	82	0.0054	103
0.10	0.0035	66	0.0043	83	0.0052	99	0.0065	124
0.12	0.0040	76	0.0050	95	0.0061	114	0.0076	143
0.14	0.0046	85	0.0057	107	0.0068	128	0.0085	160
0.16	0.0050	93	0.0063	117	0.0076	140	0.0094	175
0.18	0.0055	101	0.0068	126	0.0082	151	0.0103	189
0.20	0.0059	107	0.0073	134	0.0088	161	0.0110	201
0.22	0.0062	113	0.0078	141	0.0093	169	0.0117	211
0.24	0.0065	117	0.0082	147	0.0098	176	0.0122	220
0.26	0.0068	121	0.0085	152	0.0102	182	0.0127	228
0.28	0.0070	125	0.0088	156	0.0105	187	0.0132	234
0.30	0.0072	128	0.0090	160	0.0108	192	0.0135	240
0.32	0.0073	129	0.0092	161	0.0110	194	0.0138	242
0.34	0.0074	130	0.0093	163	0.0112	196	0.0139	245
0.36	0.0075	131	0.0094	164	0.0112	197	0.0140	246
$k = 0.375$	0.0075	131	0.0094	164	0.0113	197	0.0140	246

$k = 0.375$ for all of the above cases.

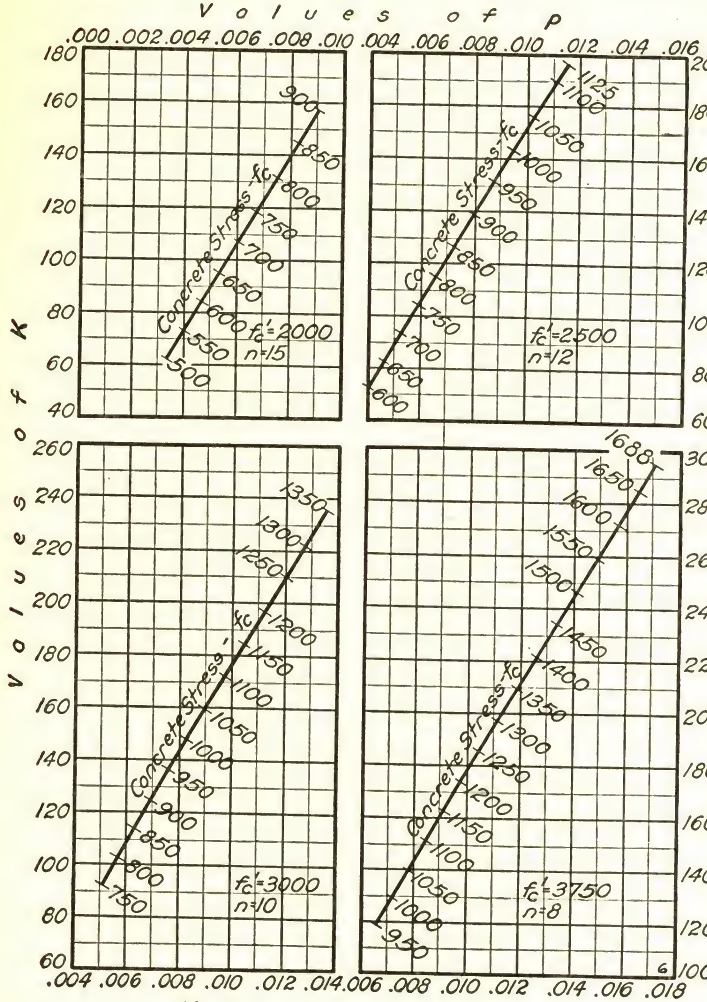
INSTRUCTIONS FOR USE.—The compressive stress in the stem of the beam, between the neutral axis and the lower face of the flange is not made use of in the table. Where many T-beams are involved having small values of t/d and wide stems, this additional strength may be taken in account in design.

GENERAL NOTE FOR TABLES 3–19 INCLUSIVE

The resisting moment of a beam with balanced reinforcement is equal to Kbd^2. The area of the tensile reinforcement is equal to pbd. The area of the compressive reinforcement, where used, is equal to $p'bd$ A beam with balanced reinforcement is one in which the full tensile and compressive unit stresses shown in the tables are developed. For steps to be taken in design of beams consult text.

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

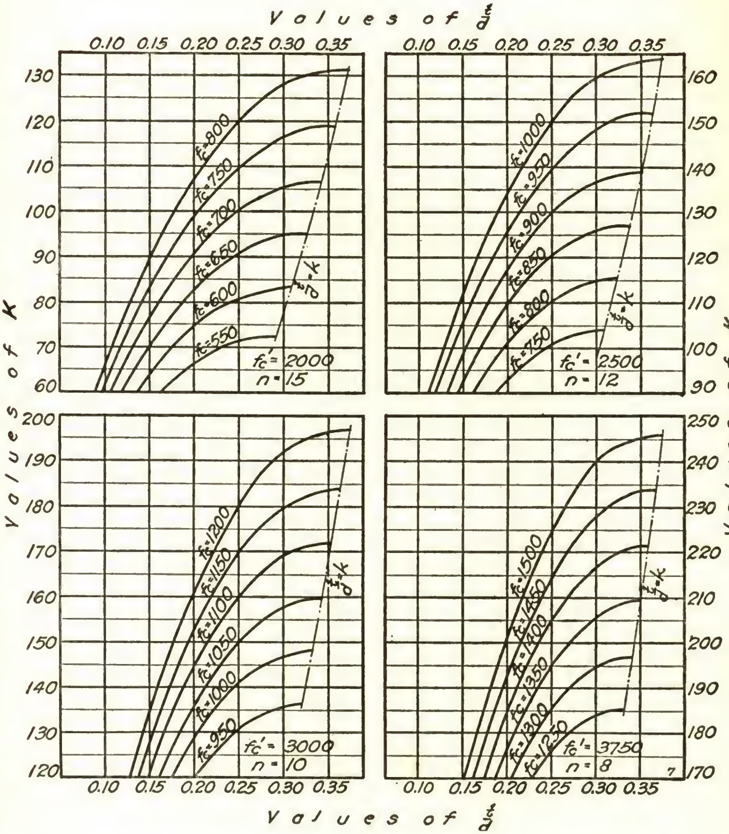
DIAGRAM 6.—RECTANGULAR BEAMS. f_c VARIES



INSTRUCTIONS FOR USE.—Use this diagram to determine the actual concrete stress (which is less than the maximum allowable value with balanced reinforcement), when the steel area has been determined by formula (103b) as described on page 6. Determine the value of K by formula (104), page 10, and read off the concrete stress at the intersection of a horizontal line through the value of K with the sloping scale of stresses.

DESIGN VALUES OF p AND K FOR $f_s = 20,000$

DIAGRAM 7.—TEE BEAMS. f_c VARIES



INSTRUCTIONS FOR USE.—Use this diagram to determine the actual stress (which is less than the maximum allowable value with balanced reinforcement), when the steel area has been determined by formula (103b) as described on page 6. Determine the value of K by formula (104), page 10, and the value of t/d from the known slab thickness and assumed beam depth. Read off the concrete stress between the curved scales at the intersection of a vertical line through the value of t/d with a horizontal line through the value of K.

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 8.—RECTANGULAR BEAMS WITH COMPRESSIVE REINFORCEMENT.

$fc = 0.40f'c$

p'	2000-lb. Concrete $n = 15$ $fc = 800$									
	$d'/d = 0.02$		$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0086	152	0.0085	150	0.0084	149	0.0084	147	0.0083	146
0.004..........	0.0096	173	0.0095	169	0.0094	166	0.0093	163	0.0092	161
0.006..........	0.0107	193	0.0105	189	0.0103	184	0.0101	180	0.0100	175
0.008..........	0.0117	214	0.0115	208	0.0113	202	0.0110	196	0.0108	180
0.010..........	0.0128	235	0.0125	227	0.0122	219	0.0119	212	0.0116	205
0.012..........	0.0139	256	0.0135	246	0.0131	237	0.0128	228	0.0124	220
0.014..........	0.0149	276	0.0145	265	0.0141	255	0.0137	244	0.0132	234
0.016..........	0.0160	297	0.0155	285	0.0150	272	0.0145	261	0.0141	249
0.018..........	0.0170	318	0.0165	304	0.0160	290	0.0154	277	0.0149	264
0.020..........	0.0181	339	0.0175	323	0.0169	308	0.0163	293	0.0157	279
0.022..........	0.0192	360	0.0185	342	0.0178	325	0.0172	309	0.0165	294
0.024..........	0.0202	380	0.0195	361	0.0188	343	0.0181	325	0.0174	308
0.026..........	0.0213	401	0.0205	381	0.0197	361	0.0189	342	0.0182	323
0.028..........	0.0223	422	0.0215	400	0.0207	379	0.0198	358	0.0190	338
0.030..........	0.0234	442	0.0225	419	0.0216	396	0.0207	374	0.0198	353

p'	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0083	144	0.0082	143	0.0081	142	0.0081	141	0.0080	139
0.004..........	0.0090	158	0.0089	155	0.0088	153	0.0087	150	0.0085	148
0.006..........	0.0098	171	0.0096	167	0.0094	163	0.0092	160	0.0091	156
0.008..........	0.0105	185	0.0103	179	0.0101	174	0.0098	169	0.0096	164
0.010..........	0.0113	198	0.0110	191	0.0107	185	0.0104	179	0.0101	173
0.012..........	0.0121	211	0.0117	203	0.0113	196	0.0110	188	0.0106	181
0.014..........	0.0128	225	0.0124	216	0.0120	207	0.0116	198	0.0112	190
0.016..........	0.0136	238	0.0131	228	0.0126	217	0.0122	207	0.0117	198
0.018..........	0.0143	252	0.0138	240	0.0133	228	0.0127	217	0.0122	206
0.020..........	0.0151	265	0.0145	252	0.0139	239	0.0133	226	0.0127	215
0.022..........	0.0159	278	0.0152	264	0.0145	250	0.0139	236	0.0132	223
0.024..........	0.0166	292	0.0159	276	0.0152	261	0.0145	246	0.0138	231
0.026..........	0.0174	305	0.0166	288	0.0158	271	0.0151	255	0.0143	240
0.028..........	0.0181	319	0.0173	300	0.0165	282	0.0157	265	0.0148	248
0.030..........	0.0189	332	0.0180	312	0.0171	293	0.0162	274	0.0153	256

INSTRUCTIONS FOR USE.—This table is to be used in beams and slabs made of 2000-lb. concrete and having compressive reinforcement, except for the special case covered by Table 9. (See also general note under Table 5.)

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 9.—RECTANGULAR BEAMS WITH COMPRESSIVE REINFORCEMENT. $f_c = 0.45f'_c$

2000-lb. Concrete $n = 15$ $f_c = 900$

p'	$d'/d = 0.02$		$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0103	180	0.0102	179	0.0102	177	0.0101	176	0.0100	174
0.004..........	0.0115	204	0.0114	200	0.0112	197	0.0111	194	0.0110	191
0.006..........	0.0127	227	0.0125	222	0.0123	218	0.0121	213	0.0119	208
0.008..........	0.0139	251	0.0136	244	0.0134	238	0.0131	231	0.0129	225
0.010..........	0.0151	274	0.0148	266	0.0145	258	0.0141	250	0.0138	242
0.012..........	0.0163	298	0.0159	288	0.0155	278	0.0151	268	0.0148	259
0.014..........	0.0175	321	0.0170	310	0.0166	298	0.0161	287	0.0157	276
0.016..........	0.0187	345	0.0181	331	0.0177	319	0.0172	306	0.0167	293
0.018..........	0.0199	368	0.0193	353	0.0188	339	0.0182	324	0.0176	310
0.020..........	0.0211	392	0.0204	375	0.0198	359	0.0192	343	0.0186	328
0.022..........	0.0223	415	0.0215	397	0.0209	379	0.0202	361	0.0195	345
0.024..........	0.0235	438	0.0226	419	0.0219	399	0.0212	380	0.0205	362
0.026..........	0.0247	462	0.0238	440	0.0230	419	0.0222	398	0.0214	379
0.028..........	0.0259	485	0.0249	462	0.0241	439	0.0232	417	0.0224	396
0.030..........	0.0271	509	0.0261	484	0.0252	459	0.0242	436	0.0233	413

p'	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0100	173	0.0099	171	0.0099	170	0.0098	168	0.0097	167
0.004..........	0.0109	188	0.0107	185	0.0106	182	0.0105	180	0.0104	177
0.006..........	0.0117	204	0.0116	199	0.0114	195	0.0112	191	0.0110	187
0.008..........	0.0126	219	0.0124	214	0.0121	208	0.0119	203	0.0116	198
0.010..........	0.0135	235	0.0132	228	0.0129	221	0.0126	214	0.0123	208
0.012..........	0.0144	250	0.0140	242	0.0136	234	0.0133	226	0.0129	218
0.014..........	0.0153	266	0.0148	256	0.0144	246	0.0140	237	0.0135	228
0.016..........	0.0161	282	0.0157	270	0.0152	259	0.0147	249	0.0142	238
0.018..........	0.0170	297	0.0165	284	0.0159	272	0.0154	260	0.0148	248
0.020..........	0.0179	313	0.0173	298	0.0167	285	0.0161	271	0.0154	259
0.022..........	0.0188	328	0.0181	312	0.0175	297	0.0168	283	0.0161	269
0.024..........	0.0197	344	0.0189	327	0.0182	310	0.0175	294	0.0167	279
0.026..........	0.0206	359	0.0197	341	0.0190	323	0.0182	306	0.0173	289
0.028..........	0.0215	375	0.0206	355	0.0197	336	0.0189	317	0.0180	299
0.030..........	0.0224	391	0.0214	369	0.0205	349	0.0196	329	0.0186	309

INSTRUCTIONS FOR USE.—This table is to be used only at sections adjacent to the supports of continuous or fixed beams or of rigid frames, made of 2000-lb. concrete and having compressive reinforcement. (See also general note under Table 5.)

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 10.—RECTANGULAR BEAMS WITH COMPRESSIVE REINFORCEMENT. $f_c = 0.40f'_c$

p'	2500-lb. Concrete $n = 12$ $f_c = 1000$									
	$d'/d = 0.02$		$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0104	185	0.0104	183	0.0103	181	0.0103	180	0.0102	179
0.004..........	0.0115	205	0.0114	202	0.0112	199	0.0111	196	0.0110	193
0.006..........	0.0125	225	0.0123	221	0.0122	216	0.0120	212	0.0118	208
0.008..........	0.0136	246	0.0133	239	0.0131	234	0.0129	228	0.0126	222
0.010..........	0.0146	266	0.0143	258	0.0140	251	0.0137	244	0.0134	237
0.012..........	0.0156	286	0.0153	277	0.0149	268	0.0146	260	0.0142	251
0.014..........	0.0167	307	0.0163	296	0.0159	286	0.0155	275	0.0150	266
0.016..........	0.0177	327	0.0172	314	0.0168	303	0.0163	291	0.0159	280
0.018..........	0.0188	348	0.0182	334	0.0177	321	0.0172	307	0.0167	295
0.020..........	0.0198	368	0.0192	352	0.0186	338	0.0181	323	0.0175	309
0.022..........	0.0208	388	0.0202	371	0.0196	355	0.0189	339	0.0183	324
0.024..........	0.0219	408	0.0212	390	0.0205	373	0.0198	355	0.0191	338
0.026..........	0.0229	429	0.0221	409	0.0214	390	0.0207	371	0.0199	353
0.028..........	0.0239	449	0.0231	428	0.0223	408	0.0215	387	0.0207	367
0.030..........	0.0250	470	0.0241	447	0.0233	425	0.0224	403	0.0215	382

p'	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0101	177	0.0101	176	0.0100	175	0.0100	173	0.0099	172
0.004..........	0.0109	190	0.0108	188	0.0107	185	0.0105	183	0.0104	180
0.006..........	0.0116	203	0.0115	200	0.0113	196	0.0111	192	0.0109	189
0.008..........	0.0124	217	0.0122	211	0.0119	206	0.0117	201	0.0115	197
0.010..........	0.0131	230	0.0128	223	0.0126	217	0.0123	211	0.0120	205
0.012..........	0.0139	243	0.0135	235	0.0132	227	0.0128	220	0.0125	213
0.014..........	0.0146	256	0.0142	247	0.0138	238	0.0134	230	0.0130	221
0.016..........	0.0154	269	0.0149	259	0.0144	249	0.0140	239	0.0135	230
0.018..........	0.0161	283	0.0156	271	0.0151	259	0.0145	248	0.0140	238
0.020..........	0.0169	295	0.0163	283	0.0157	270	0.0151	258	0.0145	246
0.022..........	0.0176	309	0.0170	295	0.0163	281	0.0157	267	0.0150	254
0.024..........	0.0184	322	0.0177	306	0.0170	291	0.0163	276	0.0156	263
0.026..........	0.0191	335	0.0184	318	0.0176	302	0.0168	285	0.0161	271
0.028..........	0.0199	349	0.0191	330	0.0182	312	0.0174	295	0.0166	279
0.030..........	0.0206	362	0.0197	342	0.0189	323	0.0180	304	0.0171	287

INSTRUCTIONS FOR USE.—This table is to be used in beams and slabs made of 2500-lb. concrete and having compressive reinforcement, except for the special case covered by Table 11. (See also general note under Table 5.)

DESIGN VALUES OF p AND K FOR $fs = 20{,}000$

TABLE 11.—RECTANGULAR BEAMS WITH COMPRESSIVE REINFORCEMENT.

$fc = 0.45f'c$

p'	2500-lb. Concrete $n = 12$ $fc = 1125$									
	$d'/d = 0.02$		$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0125	219	0.0124	217	0.0124	216	0.0123	214	0.0122	213
0.004..........	0.0137	242	0.0135	238	0.0134	236	0.0133	232	0.0132	229
0.006..........	0.0148	265	0.0146	260	0.0145	255	0.0143	251	0.0141	246
0.008..........	0.0160	288	0.0158	282	0.0155	275	0.0153	269	0.0150	263
0.010..........	0.0172	311	0.0169	303	0.0166	295	0.0163	287	0.0160	280
0.012..........	0.0183	334	0.0180	324	0.0176	315	0.0172	305	0.0169	296
0.014..........	0.0195	357	0.0191	346	0.0187	335	0.0182	324	0.0178	313
0.016..........	0.0207	380	0.0202	367	0.0197	354	0.0192	342	0.0187	330
0.018..........	0.0219	403	0.0213	389	0.0208	374	0.0202	360	0.0197	347
0.020..........	0.0230	426	0.0224	410	0.0218	394	0.0212	378	0.0206	363
0.022..........	0.0242	450	0.0236	431	0.0229	414	0.0222	397	0.0215	380
0.024..........	0.0254	473	0.0247	452	0.0239	434	0.0232	415	0.0225	397
0.026..........	0.0266	496	0.0258	474	0.0250	453	0.0242	433	0.0234	414
0.028..........	0.0277	519	0.0269	496	0.0260	473	0.0252	451	0.0243	430
0.030..........	0.0289	542	0.0280	517	0.0271	493	0.0262	470	0.0253	447

p'	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0122	211	0.0121	210	0.0120	209	0.0120	207	0.0119	206
0.004..........	0.0130	227	0.0129	224	0.0128	221	0.0127	218	0.0125	216
0.006..........	0.0139	242	0.0137	238	0.0135	234	0.0133	230	0.0132	226
0.008..........	0.0148	257	0.0145	252	0.0143	246	0.0140	241	0.0138	236
0.010..........	0.0156	272	0.0153	265	0.0150	259	0.0147	252	0.0144	246
0.012..........	0.0165	288	0.0161	279	0.0157	271	0.0154	263	0.0150	256
0.014..........	0.0174	303	0.0169	293	0.0165	284	0.0161	275	0.0157	266
0.016..........	0.0183	318	0.0178	307	0.0172	296	0.0167	286	0.0163	276
0.018..........	0.0192	334	0.0186	321	0.0180	309	0.0174	297	0.0169	286
0.020..........	0.0200	349	0.0194	335	0.0187	321	0.0181	308	0.0175	296
0.022..........	0.0208	364	0.0202	349	0.0194	334	0.0188	320	0.0182	306
0.024..........	0.0216	379	0.0210	363	0.0202	346	0.0195	331	0.0188	316
0.026..........	0.0225	395	0.0218	376	0.0209	359	0.0202	342	0.0194	326
0.028..........	0.0234	410	0.0226	390	0.0217	371	0.0209	353	0.0200	336
0.030..........	0.0243	425	0.0234	404	0.0225	384	0.0216	364	0.0206	346

INSTRUCTIONS FOR USE.—This table is to be used only at sections adjacent to the supports of continuous or fixed beams or of rigid frames, made of 2500-lb. concrete and having compressive reinforcement. (See also general note under Table 5.)

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 12.—RECTANGULAR BEAMS WITH COMPRESSIVE REINFORCEMENT.

$f_c = 0.40f'_c$

3000-lb. Concrete $n = 10$ $f_c = 1200$

p'	$d'/d = 0.02$		$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0122	217	0.0122	216	0.0121	214	0.0120	213	0.0120	211
0.004..........	0.0132	237	0.0131	234	0.0130	231	0.0129	228	0.0128	226
0.006..........	0.0143	257	0.0141	252	0.0139	248	0.0137	244	0.0136	240
0.008..........	0.0153	277	0.0151	271	0.0148	265	0.0146	260	0.0144	254
0.010..........	0.0163	297	0.0160	290	0.0158	282	0.0154	275	0.0152	268
0.012..........	0.0173	317	0.0170	308	0.0167	299	0.0163	291	0.0160	283
0.014..........	0.0183	337	0.0179	327	0.0176	316	0.0171	306	0.0167	297
0.016..........	0.0194	357	0.0189	345	0.0185	333	0.0180	322	0.0175	311
0.018..........	0.0204	377	0.0198	364	0.0194	350	0.0188	338	0.0183	325
0.020..........	0.0214	397	0.0208	382	0.0203	367	0.0197	353	0.0191	340
0.022..........	0.0224	417	0.0218	401	0.0212	385	0.0205	369	0.0199	354
0.024..........	0.0234	437	0.0227	419	0.0221	402	0.0214	385	0.0207	368
0.026..........	0.0245	457	0.0237	438	0.0230	419	0.0222	400	0.0215	382
0.028..........	0.0255	477	0.0247	456	0.0239	436	0.0231	416	0.0223	397
0.030..........	0.0265	497	0.0257	475	0.0248	453	0.0239	432	0.0231	411

p'	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0119	210	0.0119	209	0.0118	207	0.0118	206	0.0117	205
0.004..........	0.0127	223	0.0125	220	0.0124	218	0.0123	215	0.0122	213
0.006..........	0.0134	236	0.0132	232	0.0131	228	0.0129	225	0.0127	221
0.008..........	0.0141	249	0.0139	243	0.0137	239	0.0134	234	0.0132	229
0.010..........	0.0149	262	0.0146	255	0.0143	249	0.0140	243	0.0137	237
0.012..........	0.0156	275	0.0153	267	0.0149	259	0.0146	252	0.0142	245
0.014..........	0.0163	287	0.0159	278	0.0155	270	0.0151	261	0.0147	253
0.016..........	0.0171	300	0.0166	290	0.0162	280	0.0157	271	0.0152	262
0.018..........	0.0178	313	0.0173	302	0.0168	291	0.0162	280	0.0157	270
0.020..........	0.0185	326	0.0180	313	0.0174	301	0.0168	289	0.0162	278
0.022..........	0.0193	339	0.0186	325	0.0180	311	0.0174	298	0.0167	286
0.024..........	0.0200	352	0.0193	337	0.0186	322	0.0179	307	0.0172	294
0.026..........	0.0207	365	0.0200	348	0.0191	332	0.0185	317	0.0178	302
0.028..........	0.0215	378	0.0207	360	0.0199	343	0.0191	326	0.0183	310
0.030..........	0.0222	391	0.0214	372	0.0205	353	0.0196	335	0.0188	318

INSTRUCTIONS FOR USE.—This table is to be used in beams and slabs made of 3000-lb. concrete and having compressive reinforcement except for the special case covered by Table 13. (See also general note under Table 5.)

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 13.—RECTANGULAR BEAMS WITH COMPRESSIVE REINFORCEMENT. $f_c = 0.45 f'_c$

p'	3000-lb. Concrete $n = 10$ $f_c = 1350$									
	$d'/d = 0.02$		$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0148	258	0.0147	256	0.0146	254	0.0146	253	0.0145	251
0.004..........	0.0159	280	0.0158	277	0.0157	274	0.0155	271	0.0154	268
0.006..........	0.0171	303	0.0169	298	0.0167	293	0.0165	289	0.0163	284
0.008..........	0.0182	325	0.0180	319	0.0177	313	0.0175	307	0.0173	301
0.010..........	0.0194	348	0.0191	340	0.0188	332	0.0185	325	0.0182	317
0.012..........	0.0205	371	0.0201	361	0.0198	352	0.0194	342	0.0191	334
0.014..........	0.0217	393	0.0212	382	0.0208	371	0.0204	360	0.0200	350
0.016..........	0.0228	416	0.0223	403	0.0218	391	0.0214	378	0.0209	367
0 018..........	0.0240	439	0.0234	424	0.0229	410	0.0223	396	0.0218	383
0.020..........	0.0251	461	0.0245	445	0.0239	429	0.0233	414	0.0227	399
0.022..........	0.0263	484	0.0256	466	0.0249	449	0.0243	432	0.0236	416
0.024..........	0.0274	506	0.0267	487	0.0260	468	0.0252	450	0.0246	432
0.026..........	0.0286	529	0.0278	508	0.0270	488	0.0262	468	0.0255	449
0.028..........	0.0297	552	0.0289	529	0.0280	507	0.0272	485	0.0264	465
0.030..........	0.0309	574	0.0300	550	0.0291	527	0.0282	504	0.0273	482

p'	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0145	250	0.0144	249	0.0143	247	0.0143	246	0.0142	245
0.004..........	0.0153	265	0.0152	262	0.0151	260	0.0149	257	0.0148	255
0.006..........	0.0162	280	0.0160	276	0.0158	272	0.0156	268	0.0154	264
0 008..........	0.0170	295	0.0168	290	0.0165	284	0.0163	279	0.0160	274
0.010..........	0.0179	310	0.0176	303	0.0173	297	0.0170	290	0.0167	284
0.012..........	0.0187	325	0.0183	317	0.0180	309	0.0176	301	0.0173	294
0.014....	0.0196	340	0.0191	330	0.0187	321	0.0183	312	0.0179	304
0.016..........	0.0204	355	0.0199	344	0.0194	333	0.0190	323	0.0185	313
0.018..........	0.0213	370	0.0207	358	0.0202	346	0.0196	334	0.0191	323
0.020..........	0.0221	385	0.0215	371	0.0209	358	0.0203	345	0.0197	333
0.022..........	0.0230	400	0.0223	385	0.0216	370	0.0210	356	0.0203	343
0.024..........	0.0238	415	0.0231	399	0.0224	383	0.0216	367	0.0209	353
0.026..........	0.0247	430	0.0239	412	0.0231	395	0.0223	378	0.0216	362
0.028..........	0.0255	445	0.0247	426	0.0238	407	0.0230	389	0.0222	372
0.030..........	0.0264	460	0.0255	440	0.0246	420	0.0237	400	0.0228	382

INSTRUCTIONS FOR USE.—This table is to be used only at sections adjacent to the supports of continuous or fixed beams or of rigid frames, made of 3000-lb. concrete and having compressive reinforcement. (See also general note under Table 5.)

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 14.—RECTANGULAR BEAMS WITH COMPRESSIVE REINFORCEMENT. $f_c = 0.40 f'_c$

3750-lb. Concrete $n = 8$ $f_c = 1500$

p'	$d'/d = 0.02$		$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	p	K	p	K	p	K	p	K	p	K
0.002	0.0151	265	0.0150	264	0.0150	263	0.0149	261	0.0149	260
0.004	0.0161	285	0.0160	282	0.0159	279	0.0158	276	0.0156	274
0.006	0.0171	304	0.0169	300	0 0167	296	0.0166	292	0.0164	288
0.008	0.0181	324	0.0179	318	0.0176	312	0.0174	307	0.0172	301
0.010	0.0191	343	0.0188	336	0.0185	329	0.0182	322	0.0179	315
0.012	0.0201	363	0.0197	354	0.0194	346	0.0191	337	0.0187	329
0.014	0.0211	382	0.0207	372	0.0203	362	0.0199	352	0.0195	343
0.016	0.0221	402	0.0216	390	0.0212	379	0.0207	368	0.0203	357
0.018	0.0230	421	0.0225	408	0.0220	395	0.0215	383	0.0210	371
0.020	0.0240	441	0.0235	426	0.0229	412	0.0224	398	0.0218	385
0.022	0.0250	460	0.0244	444	0.0238	429	0.0232	413	0.0226	398
0.024	0.0260	480	0.0253	462	0.0247	445	0.0240	428	0.0233	412
0.026	0.0270	499	0.0263	480	0.0256	462	0.0248	444	0.0241	426
0.028	0.0280	518	0.0272	498	0.0265	478	0.0257	459	0.0249	440
0.030	0.0290	538	0.0282	516	0.0273	495	0.0265	474	0.0256	454

p'	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	p	K	p	K	p	K	p	K	p	K
0.002	0.0148	259	0.0148	257	0.0147	256	0.0146	255	0.0146	254
0.004	0.0155	271	0.0154	269	0.0153	266	0.0152	264	0.0151	262
0.006	0.0162	284	0.0161	280	0.0159	276	0.0157	273	0.0156	270
0.008	0.0170	296	0.0167	291	0.0165	286	0.0163	282	0.0161	277
0.010	0.0177	309	0.0174	303	0.0171	297	0.0168	291	0.0166	285
0.012	0.0184	321	0.0180	314	0.0177	307	0.0174	300	0.0170	293
0.014	0.0191	334	0.0187	325	0.0183	317	0.0179	309	0.0175	301
0.016	0.0198	346	0.0194	337	0.0189	327	0.0185	318	0.0180	309
0.018	0.0205	359	0.0200	348	0.0195	337	0.0190	327	0.0185	317
0.020	0.0212	372	0.0207	359	0.0201	347	0.0196	336	0.0190	324
0.022	0.0220	384	0.0213	371	0.0207	357	0.0201	345	0.0195	332
0.024	0.0227	397	0.0220	382	0.0213	367	0.0207	354	0.0200	340
0.026	0.0234	409	0.0227	393	0.0219	378	0.0212	362	0.0205	348
0.028	0.0241	422	0.0233	404	0.0225	388	C.0217	371	0.0210	356
0.030	0.0248	434	0.0240	416	0.0231	398	0.0223	380	0.0214	364

INSTRUCTIONS FOR USE.—This table is to be used in beams and slabs made of 3750-lb. concrete and having compressive reinforcement, except for the special case covered by Table 15. (See also general note under Table 5.)

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 15.—RECTANGULAR BEAMS WITH COMPRESSIVE REINFORCEMENT. $f_c = 0.45 f'_c$

p'	3750-lb. Concrete $n = 8$ $f_c = 1688$									
	$d'/d = 0.02$		$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	p	K	p	K	p	K	p	K	p	K
0.002	0.0181	316	0.0181	314	0.0180	313	0.0180	311	0.0179	310
0.004	0.0192	338	0.0191	335	0.0190	332	0.0189	329	0.0188	326
0.006	0.0204	360	0.0202	355	0.0200	351	0.0198	346	0.0197	342
0 008	0.0215	382	0.0212	376	0.0210	370	0.0208	364	0.0206	358
0.010	0.0226	404	0.0223	396	0.0220	388	0.0217	381	0.0214	374
0.012	0.0237	426	0.0234	416	0.0230	407	0.0226	399	0.0223	390
0.014	0.0249	448	0.0244	437	0.0240	426	0.0236	416	0.0232	406
0.016	0.0260	470	0.0255	457	0.0250	445	0.0245	433	0.0241	422
0.018	0.0271	492	0.0265	478	0.0261	465	0.0255	451	0.0250	438
0.020	0.0282	514	0.0276	498	0.0271	484	0.0265	468	0.0259	454
0.022	0.0293	536	0.0287	518	0.0281	503	0.0274	486	0.0268	470
0.024	0.0304	558	0.0398	539	0.0291	522	0 0284	503	0.0277	486
0.026	0.0316	580	0.0308	559	0.0301	541	0.0293	521	0.0286	502
0.028	0.0327	602	0.0319	580	0.0311	560	0.0303	538	0.0294	518
0.030	0.0338	624	0.0330	600	0.0321	578	0.0312	556	0.0303	534

p'	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	p	K	p	K	p	K	p	K	p	K
0.002	0.0178	309	0.0178	307	0.0177	306	0.0177	305	0.0176	304
0.004	0.0187	323	0.0185	321	0.0184	318	0.0183	315	0.0182	313
0.006	0.0195	338	0.0193	334	0.0191	330	0.0189	326	0.0188	323
0.008	0.0203	352	0.0201	347	0.0199	342	0.0196	337	0.0194	332
0.010	0.0211	367	0.0208	360	0.0206	354	0.0202	348	0.0200	342
0.012	0.0220	382	0.0216	374	0.0213	366	0.0209	358	0.0206	351
0.014	0.0228	396	0.0224	387	0.0220	378	0.0215	369	0.0212	361
0.016	0.0236	410	0.0232	400	0.0227	390	0.0222	380	0.0218	370
0.018	0.0245	425	0.0339	413	0.0234	401	0.0228	390	0.0224	380
0.020	0.0253	440	0.0247	427	0.0241	414	0.0235	401	0.0230	389
0.022	0.0261	455	0.0255	440	0.0248	426	0.0242	412	0.0235	399
0.024	0.0270	469	0.0262	453	0.0255	438	0.0248	423	0.0241	408
0.026	0.0278	484	0.0270	466	0.0263	449	0.0255	433	0.0247	418
0.028	0.0286	498	0.0278	480	0.0270	461	0.0262	444	0.0253	427
0.030	0.0294	513	0.0286	493	0.0277	473	0.0268	455	0.0259	437

INSTRUCTIONS FOR USE.—This table is to be used only at sections adjacent to the supports of continuous or fixed beams or of rigid frames, made of 3750 lb. concrete and having compressive reinforcement. (See also general note under Table 5.)

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 16.—T-BEAMS WITH COMPRESSIVE REINFORCEMENT

$\frac{t}{d}$	p	K
0.04	0.0015	30
0.06	0.0022	43
0.08	0.0029	55
0.10	0.0035	66
0.12	0.0040	76
0.14	0.0046	85
0.16	0.0050	93
0.18	0.0055	101
0.20	0.0059	107
0.22	0.0062	113
0.24	0.0065	117
0.26	0.0068	121
0.28	0.0070	125
0.30	0.0072	128
0.32	0.0073	129
0.34	0.0074	130
0.36	0.0075	131
k	0.0075	131

p'	2000-lb. Concrete $n = 15$ $fc = 800$							
	$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	p	K	p	K	p	K	p	K
0.002	0.0010	19	0.0009	18	0.0009	16	0.0008	15
0.004	0.0020	38	0.0019	35	0.0018	32	0.0016	30
0.006	0.0030	58	0.0028	53	0.0026	49	0.0025	44
0.008	0.0040	77	0.0038	71	0.0035	65	0.0033	59
0.010	0.0050	96	0.0047	88	0.0044	81	0.0041	74
0.012	0.0060	115	0.0056	106	0.0053	97	0.0049	89
0.014	0.0070	134	0.0066	124	0.0062	113	0.0057	103
0.016	0.0080	154	0.0075	141	0.0070	130	0.0066	118
0.018	0.0090	173	0.0085	159	0.0079	146	0.0074	133
0.020	0.0100	192	0.0094	177	0.0088	162	0.0082	148
0.022	0.0110	211	0.0103	194	0.0097	178	0.0090	163
0.024	0.0120	230	0.0113	212	0.0106	194	0.0099	177
0.026	0.0130	250	0.0122	230	0.0114	211	0.0107	192
0.028	0.0140	269	0.0132	248	0.0123	227	0.0115	207
0.030	0.0150	288	0.0141	265	0.0132	243	0.0123	222

p'	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0008	13	0.0007	12	0.0006	11	0.0006	10	0.0005	
0.004..........	0.0015	27	0.0014	24	0.0013	21	0.0012	19	0.0010	17
0.006..........	0.0023	40	0.0021	36	0.0019	32	0.0017	29	0.0016	25
0.008..........	0.0030	54	0.0028	48	0.0026	43	0.0023	38	0.0021	33
0.010..........	0.0038	67	0.0035	60	0.0032	54	0.0029	48	0.0026	42
0.012..........	0.0046	80	0.0042	72	0.0038	65	0.0035	57	0.0031	50
0.014..........	0.0053	94	0.0049	85	0.0045	75	0.0041	67	0.0037	59
0.016..........	0.0061	107	0.0056	97	0.0051	86	0.0047	76	0.0042	67
0.018..........	0.0068	121	0.0063	109	0.0058	97	0.0052	86	0.0047	75
0.020..........	0.0076	134	0.0070	121	0.0064	108	0.0058	95	0.0052	84
0.022..........	0.0084	147	0.0077	133	0.0070	118	0.0064	105	0.0057	92
0.024..........	0.0091	161	0.0084	145	0.0077	129	0.0070	114	0.0063	100
0.026..........	0.0099	174	0.0091	157	0.0083	140	0.0075	124	0.0068	109
0.028..........	0.0106	188	0.0098	169	0.0090	151	0.0081	133	0.0073	117
0.030..........	0.0114	201	0.0105	181	0.0096	162	0.0087	143	0.0078	125

GENERAL NOTE FOR TABLES 16 TO 19

INSTRUCTIONS FOR USE.—Ordinarily T-beams require no compressive reinforcement but Tables 16 to 19 will be found useful where architectural considerations require certain T-joists or beams to carry extraordinary loads without increase in depth, or require the removal of portions of the flange. (See also general note under Table 5.)

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 17.—T-BEAMS WITH COMPRESSIVE REINFORCEMENT

2500-lb. Concrete $n = 12$ $fc = 1000$

$\frac{t}{d}$	p	K
0.04	0.0019	37
0.06	0.0028	54
0.08	0.0036	69
0.10	0.0043	83
0.12	0.0050	95
0.14	0.0057	107
0.16	0.0063	117
0.18	0.0068	126
0.20	0.0073	134
0.22	0.0078	141
0.24	0.0082	147
0.26	0.0085	152
0.28	0.0088	156
0.30	0.0090	160
0.32	0.0092	162
0.34	0.0093	163
0.36	0.0094	164
k	0.0094	164

p'	$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	p	K	p	K	p	K	p	K
0.002	0.0010	19	0.0009	17	0.0009	16	0.0008	15
0.004	0.0020	38	0.0018	35	0.0017	32	0.0016	29
0.006	0.0029	57	0.0028	52	0.0026	48	0.0024	44
0.008	0.0039	75	0.0037	70	0.0035	64	0.0032	58
0.010	0.0049	94	0.0046	87	0.0043	80	0.0040	73
0.012	0.0059	113	0.0055	104	0.0052	96	0.0048	87
0.014	0.0069	132	0.0065	122	0.0061	112	0.0056	102
0.016	0.0079	151	0.0074	139	0.0069	127	0.0065	116
0.018	0.0088	170	0.0083	157	0.0078	143	0.0073	131
0.020	0.0098	189	0.0092	174	0.0087	159	0.0081	145
0.022	0.0108	207	0.0102	191	0.0095	175	0.0089	160
0.024	0.0118	226	0.0111	209	0.0104	191	0.0097	174
0.026	0.0128	245	0.0120	226	0.0113	207	0.0105	189
0.028	0.0138	264	0.0129	244	0.0121	223	0.0113	203
0.030	0.0147	283	0.0139	261	0.0130	239	0.0121	218

p'	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0007	13	0.0007	12	0.0006	11	0.0006	9	0.0005	8
0.004..........	0.0015	26	0.0014	24	0.0013	21	0.0011	19	0.0010	16
0.006..........	0.0022	40	0.0021	36	0.0019	32	0.0017	28	0.0015	25
0.008..........	0.0030	53	0.0028	47	0.0025	42	0.0023	37	0.0021	33
0.010..........	0.0037	66	0.0034	59	0.0032	53	0.0029	47	0.0026	41
0.012..........	0.0045	79	0.0041	71	0.0038	64	0.0034	56	0.0031	49
0.014..........	0.0052	92	0.0048	83	0.0044	74	0.0040	66	0.0036	57
0.016..........	0.0060	105	0.0055	95	0.0050	85	0.0046	75	0.0041	66
0.018..........	0.0067	119	0.0062	107	0.0057	95	0.0051	84	0.0046	74
0.020..........	0.0075	132	0.0069	119	0.0063	106	0.0057	94	0.0051	82
0.022..........	0.0082	145	0.0076	130	0.0069	117	0.0063	103	0.0056	90
0.024..........	0.0090	158	0.0083	142	0.0076	127	0.0069	112	0.0062	99
0.026..........	0.0097	171	0.0090	154	0.0082	138	0.0074	122	0.0067	107
0.028..........	0.0105	185	0.0097	166	0.0088	148	0.0080	131	0.0072	115
0.030..........	0.0112	198	0.0103	178	0.0095	159	0.0086	140	0.0077	123

See instructions for use under Table 16.

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 18.—T-BEAMS WITH COMPRESSIVE REINFORCEMENT

3000-lb. Concrete $n = 10$ $fc = 1200$

$\frac{t}{d}$	p	K
0.04	0.0023	45
0.06	0.0033	64
0.08	0.0043	82
0.10	0.0052	99
0.12	0.0061	114
0.14	0.0068	128
0.16	0.0076	140
0.18	0.0082	151
0.20	0.0088	161
0.22	0.0093	169
0.24	0.0098	176
0.26	0.0102	182
0.28	0.0105	187
0.30	0.0108	192
0.32	0.0110	194
0.34	0.0112	196
0.36	0.0112	197
k	0.0113	197

p'	$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	p	K	p	K	p	K	p	K
0.002	0.0010	19	0.0009	17	0.0008	16	0.0008	14
0.004	0.0019	37	0.0018	34	0.0017	31	0.0016	29
0.006	0.0029	56	0.0027	51	0.0025	47	0.0024	43
0.008	0.0039	74	0.0036	68	0.0034	63	0.0032	57
0.010	0.0048	93	0.0045	85	0.0042	78	0.0040	71
0.012	0.0058	111	0.0054	102	0.0051	94	0.0048	86
0.014	0.0068	130	0.0064	119	0.0059	109	0.0056	100
0.016	0.0077	148	0.0073	136	0.0068	125	0.0063	114
0.018	0.0087	167	0.0082	154	0.0076	141	0.0071	128
0.020	0.0096	185	0.0091	171	0.0085	156	0.0079	143
0.022	0.0106	204	0.0100	188	0.0093	172	0.0087	157
0.024	0.0116	222	0.0109	205	0.0102	188	0.0095	171
0.026	0.0125	241	0.0118	222	0.0110	204	0.0103	185
0.028	0.0135	260	0.0127	239	0.0119	219	0.0111	200
0.030	0.0145	278	0.0136	256	0.0127	235	0.0119	214

p'	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0007	13	0.0007	12	0.0006	10	0.0006	9	0.0005	8
0.004..........	0.0015	26	0.0014	23	0.0012	21	0.0011	18	0.0010	16
0.006..........	0.0022	39	0.0020	35	0.0019	31	0.0017	28	0.0015	24
0.008..........	0.0029	52	0.0027	47	0.0025	42	0.0022	37	0.0020	32
0.010..........	0.0037	65	0.0034	58	0.0031	52	0.0028	46	0.0025	40
0.012..........	0.0044	78	0.0041	70	0.0037	62	0.0034	55	0.0030	48
0.014..........	0.0051	90	0.0047	81	0.0043	73	0.0039	65	0.0035	56
0.016..........	0.0059	103	0.0054	93	0.0050	83	0.0045	74	0.0040	64
0.018..........	0.0066	116	0.0061	105	0.0056	94	0.0051	83	0.0045	73
0.020..........	0.0073	129	0.0068	116	0.0062	104	0.0056	92	0.0050	81
0.022..........	0.0081	142	0.0074	128	0.0068	114	0.0062	101	0.0055	89
0.024..........	0.0088	155	0.0081	140	0.0074	125	0.0067	111	0.0060	97
0.026..........	0.0095	168	0.0088	151	0.0080	135	0.0073	120	0.0066	105
0.028..........	0.0103	181	0.0095	163	0.0087	146	0.0079	129	0.0071	113
0.030..........	0.0110	194	0.0102	175	0.0093	156	0.0084	138	0.0076	121

See instructions for use under Table 16.

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 19.—T-BEAMS WITH COMPRESSIVE REINFORCEMENT

3750-lb. Concrete $n = 8$ $f_c = 1500$

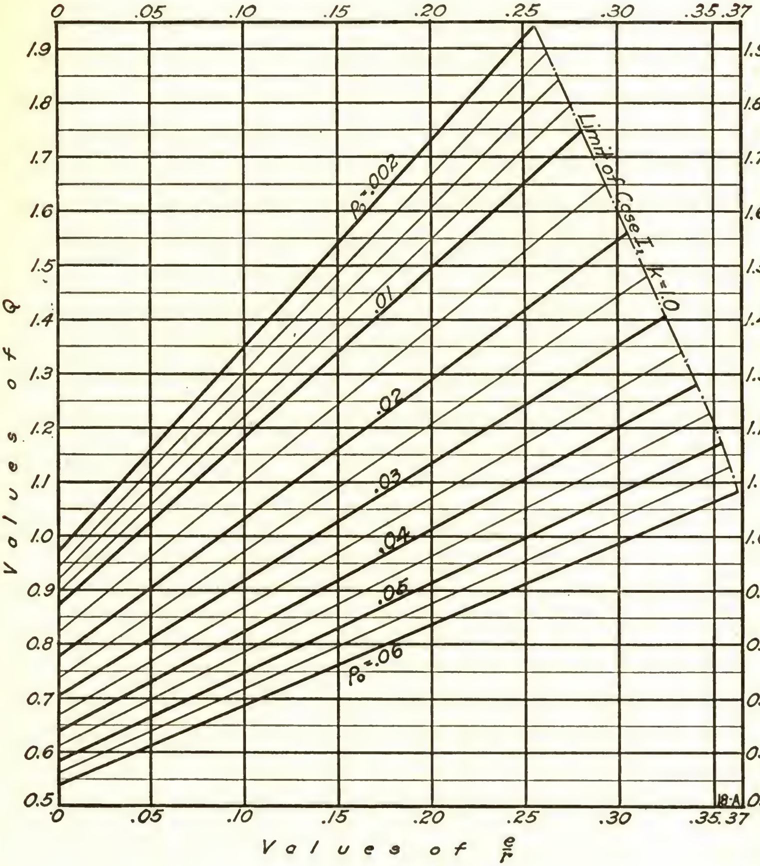
$\frac{t}{d}$	p	K	p'	$d'/d = 0.04$ p	$d'/d = 0.04$ K	$d'/d = 0.06$ p	$d'/d = 0.06$ K	$d'/d = 0.08$ p	$d'/d = 0.08$ K	$d'/d = 0.10$ p	$d'/d = 0.10$ K
0.04	0.0028	56									
0.06	0.0041	80	0.002	0.0009	18	0.0009	17	0.0008	15	0.0008	14
0.08	0.0054	103	0.004	0.0019	36	0.0018	33	0.0017	30	0.0015	28
0.10	0.0065	124	0.006	0.0028	54	0.0026	50	0.0025	46	0.0023	42
0.12	0.0076	143	0.008	0.0038	72	0.0035	66	0.0033	61	0.0031	55
0.14	0.0085	160	0.010	0.0047	90	0.0044	83	0.0041	76	0.0038	69
0.16	0.0094	175									
0.18	0.0103	189	0.012	0.0056	108	0.0053	100	0.0050	91	0.0046	83
0.20	0.0110	201	0.014	0.0066	126	0.0062	116	0.0058	106	0.0054	97
0.22	0.0117	211	0.016	0.0075	144	0.0071	133	0.0066	122	0.0062	111
0.24	0.0122	220	0.018	0.0084	162	0.0079	149	0.0074	137	0.0069	125
0.26	0.0127	228	0.020	0.0094	180	0.0088	166	0.0083	152	0.0077	139
0.28	0.0132	234									
0.30	0.0135	240	0.022	0.0103	198	0.0097	183	0.0091	167	0.0085	152
0.32	0.0138	242	0.024	0.0112	216	0.0106	199	0.0099	182	0.0092	166
0.34	0.0139	245	0.026	0.0122	234	0.0115	216	0.0107	198	0.0100	180
0.36	0.0140	246	0.028	0.0131	252	0.0124	232	0.0116	213	0.0108	194
k	0.0140	246	0.030	0.0141	270	0.0132	249	0.0124	228	0.0116	208

p'	$d'/d = 0.12$ p	$d'/d = 0.12$ K	$d'/d = 0.14$ p	$d'/d = 0.14$ K	$d'/d = 0.16$ p	$d'/d = 0.16$ K	$d'/d = 0.18$ p	$d'/d = 0.18$ K	$d'/d = 0.20$ p	$d'/d = 0.20$ K
0.002..........	0.0007	13	0.0007	11	0.0006	10	0.0005	9	0.0005	8
0.004..........	0.0014	25	0.0013	23	0.0012	20	0.0011	18	0.0010	16
0.006..........	0.0021	38	0.0020	34	0.0018	30	0.0016	27	0.0015	24
0.008..........	0.0029	50	0.0026	45	0.0024	40	0.0022	36	0.0020	31
0.010..........	0.0036	63	0.0033	57	0.0030	51	0.0027	45	0.0024	39
0.012..........	0.0043	75	0.0039	68	0.0036	61	0.0033	54	0.0029	47
0.014..........	0.0050	88	0.0046	79	0.0042	71	0.0038	63	0.0034	55
0.016..........	0.0057	100	0.0053	91	0.0048	81	0.0044	72	0.0039	63
0.018..........	0.0064	113	0.0059	102	0.0054	91	0.0049	81	0.0044	71
0.020..........	0.0071	126	0.0066	113	0.0060	101	0.0055	90	0.0049	78
0.022..........	0.0079	138	0.0072	125	0.0066	111	0.0060	99	0.0054	86
0.024..........	0.0086	151	0.0079	136	0.0072	121	0.0066	108	0.0059	94
0.026..........	0.0093	163	0.0086	147	0.0078	132	0.0071	116	0.0064	102
0.028..........	0.0100	176	0.0092	158	0.0084	142	0.0076	125	0.0069	110
0.030..........	0.0107	188	0.0099	170	0.0090	152	0.0082	134	0.0074	118

See instructions for use under Table 16.

BENDING AND DIRECT COMPRESSION—CIRCULAR SECTIONS

DIAGRAM 20.—CASE I—2000-LB. CONCRETE—$n = 15$



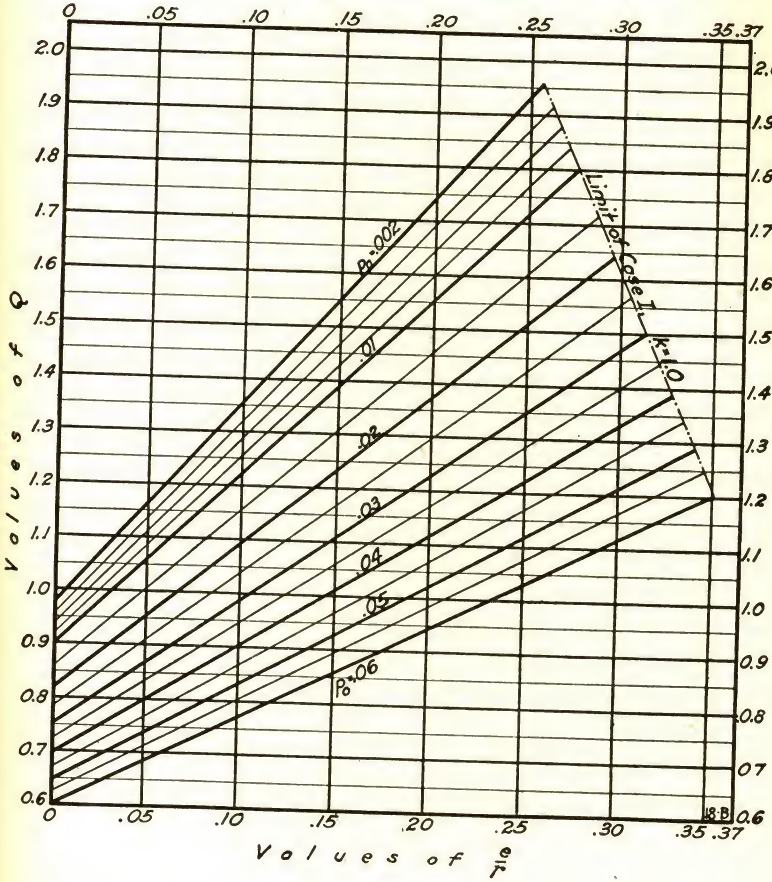
GENERAL NOTE FOR DIAGRAMS 20 TO 65

These diagrams are based on circular or rectangular sections in which the reinforcement is symmetrically placed with respect to gravity axis at right angles to the plane in which the load is eccentric.

INSTRUCTIONS FOR USE OF DIAGRAMS 20 TO 24.—Enter the diagram with the value of e/r and proceed vertically to an intersection with the sloping index line for an assumed value of p_0. From this intersection pass horizontally to the right or left marginal scale and read off the value of Q. Solve for f_c in formula (106).

BENDING AND DIRECT COMPRESSION—CIRCULAR SECTIONS

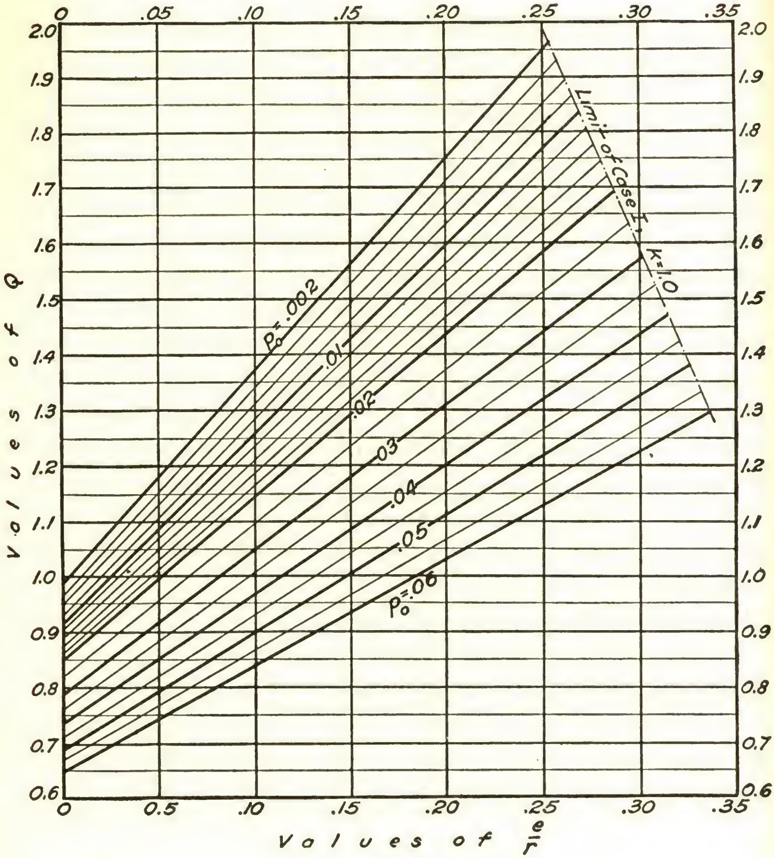
DIAGRAM 21.—CASE I—2500-LB. CONCRETE—$n = 12$



See Instructions for use and also general note under Diagram 20.

BENDING AND DIRECT COMPRESSION—CIRCULAR SECTIONS

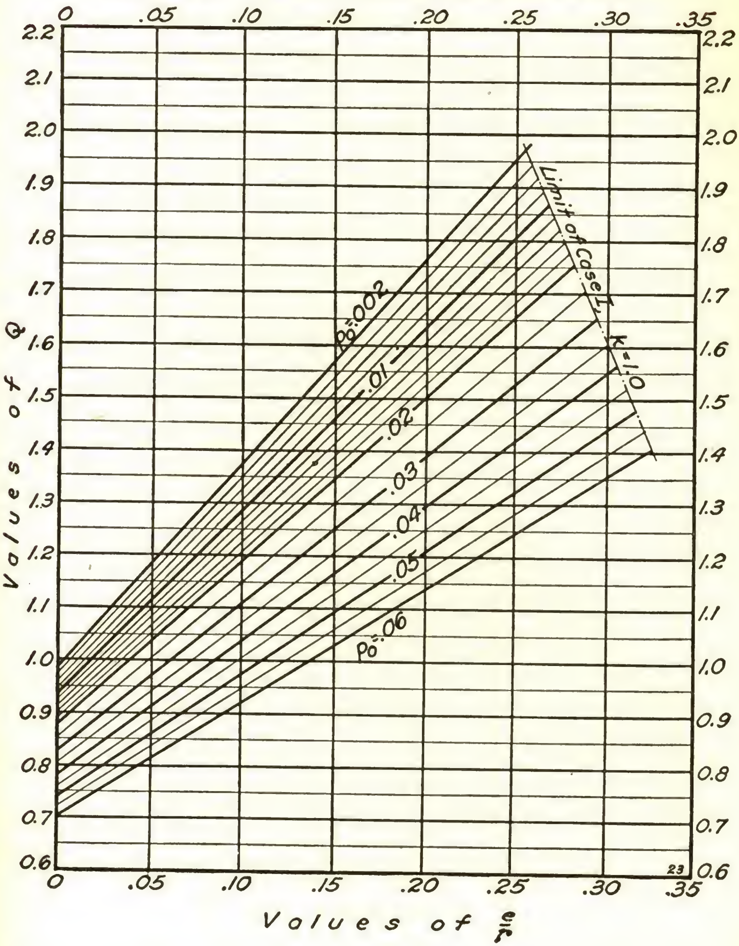
DIAGRAM 22.—CASE I—3000-LB. CONCRETE—n = 10



See instructions for use and also general note under Diagram 20.

BENDING AND DIRECT COMPRESSION—CIRCULAR SECTIONS

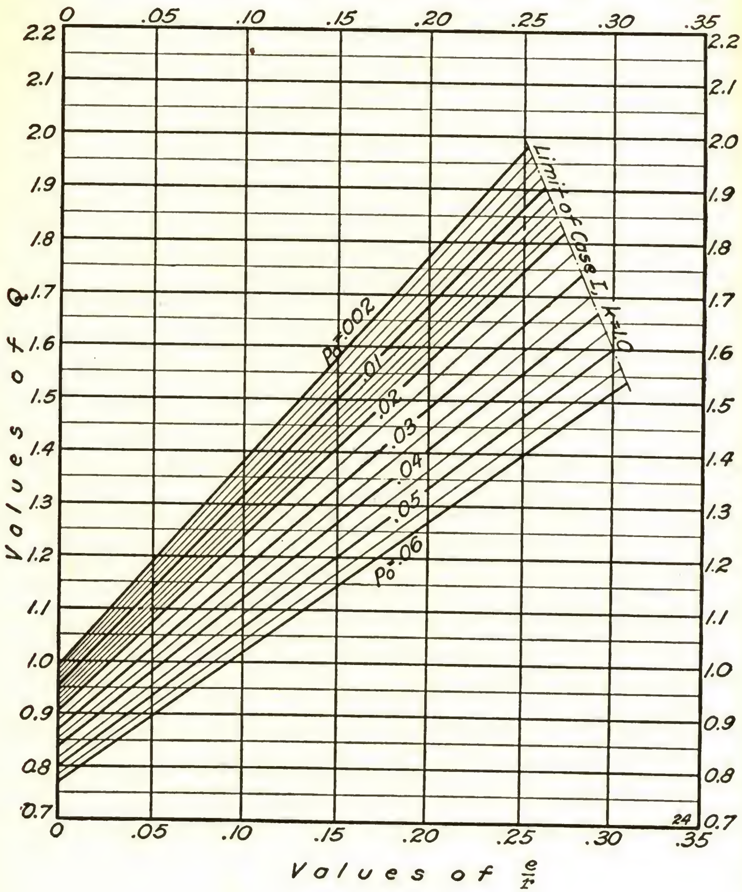
DIAGRAM 23.—CASE I—3750-LB. CONCRETE—$n = 8$



See instructions for use and also general note under Diagram 20.

BENDING AND DIRECT COMPRESSION—CIRCULAR SECTIONS

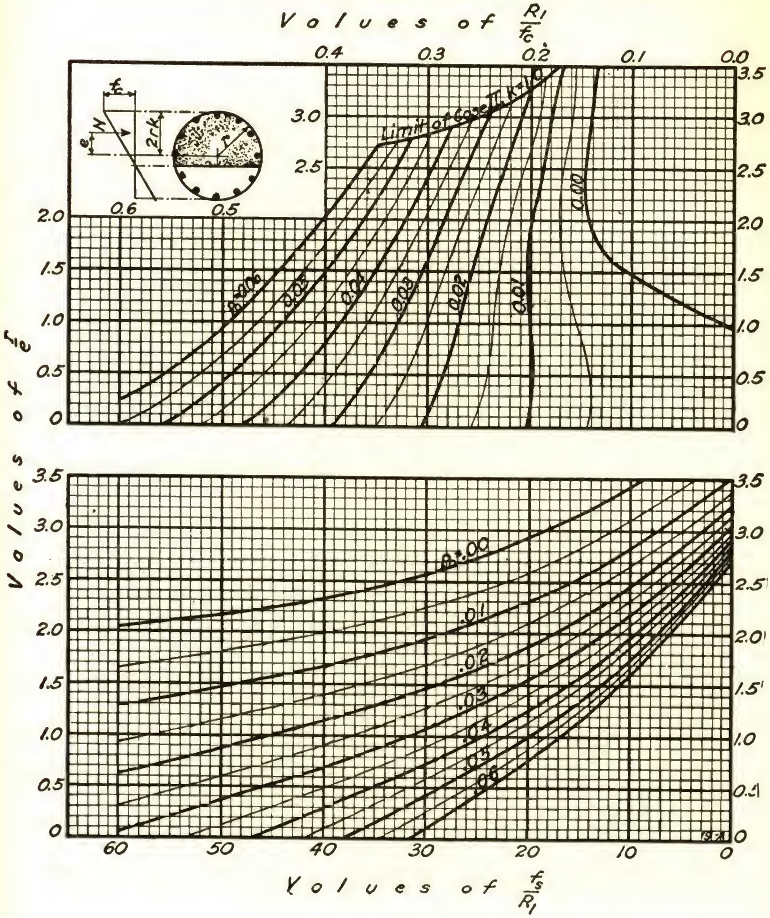
DIAGRAM 24.—CASE I—5000-LB. CONCRETE—$n = 6$



See instructions for use and also general note under Diagram 20.

BENDING AND DIRECT COMPRESSION—CIRCULAR SECTIONS

DIAGRAM 25.—CASE II—2000-LB. CONCRETE—n = 15



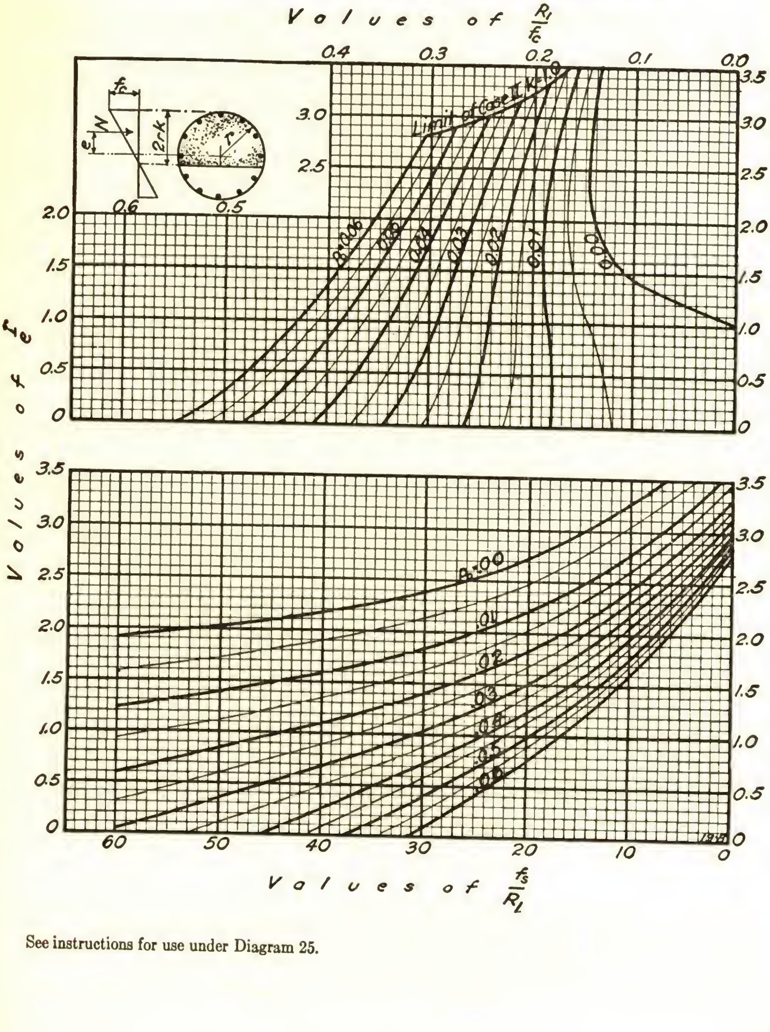
INSTRUCTIONS FOR USE OF DIAGRAMS 25 TO 29.—Enter the upper part of the diagram with the value of r/e and proceed horizontally to an intersection with the index line for an assumed value of p_o. Pass vertically to the upper marginal scale and read off the value R_1/f_c.

Enter the lower part of the diagram in the same manner. From the intersection with the p_o index line pass vertically to the lower marginal scale and read off the value of f_s/R_1.

Solve formulas (107), (108) and (109) for values of f_c and f_s. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—CIRCULAR SECTIONS

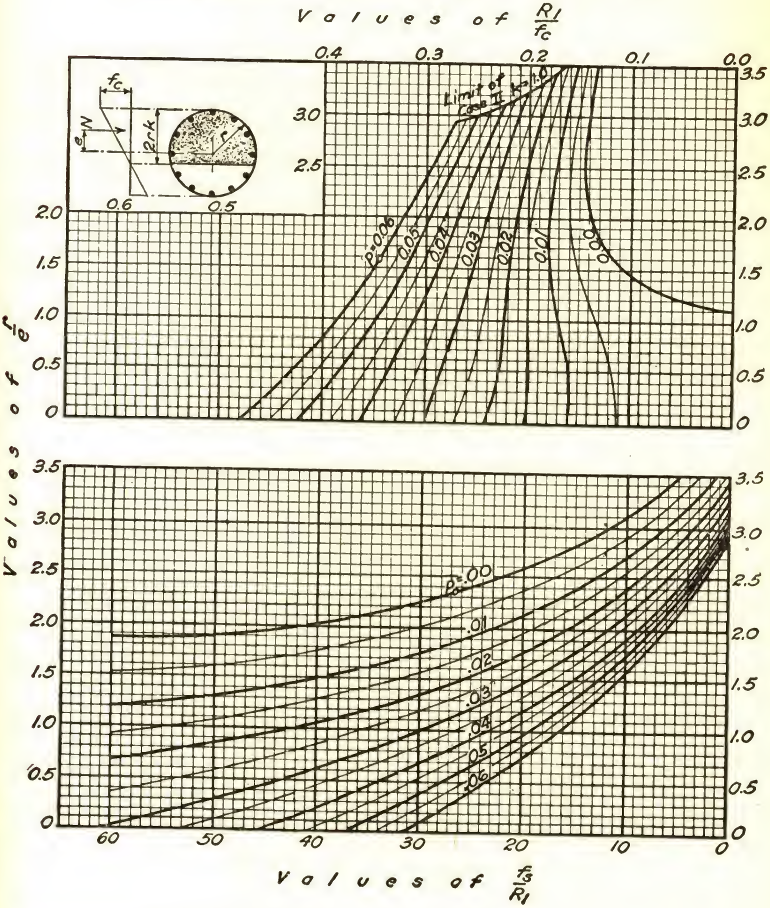
DIAGRAM 26.—CASE II—2500-LB. CONCRETE—n = 12



See instructions for use under Diagram 25.

BENDING AND DIRECT COMPRESSION—CIRCULAR SECTIONS

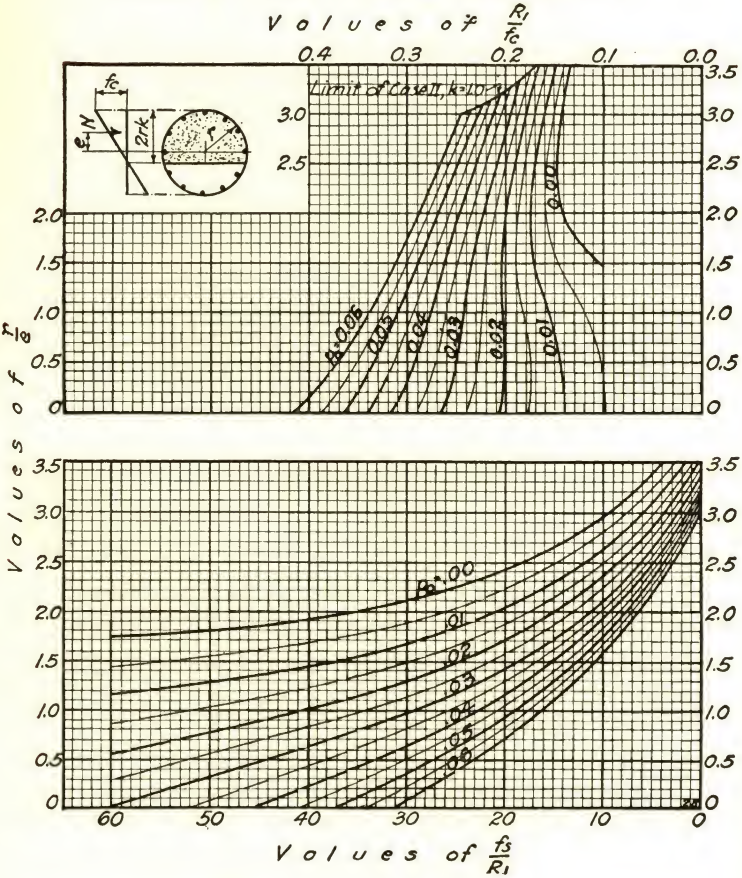
DIAGRAM 27.—CASE II—3000-LB. CONCRETE—$n = 10$



See instructions for use under Diagram 25.

BENDING AND DIRECT COMPRESSION—CIRCULAR SECTIONS

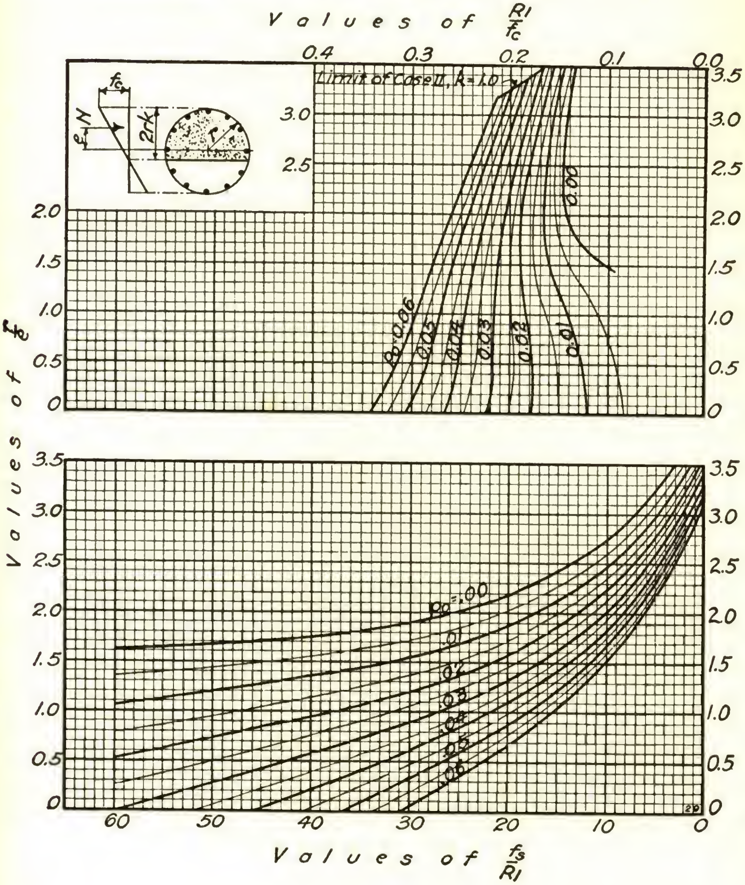
DIAGRAM 28.—CASE II—3750-LB. CONCRETE—$n = 8$



See instructions for use under Diagram 25.

BENDING AND DIRECT COMPRESSION—CIRCULAR SECTIONS

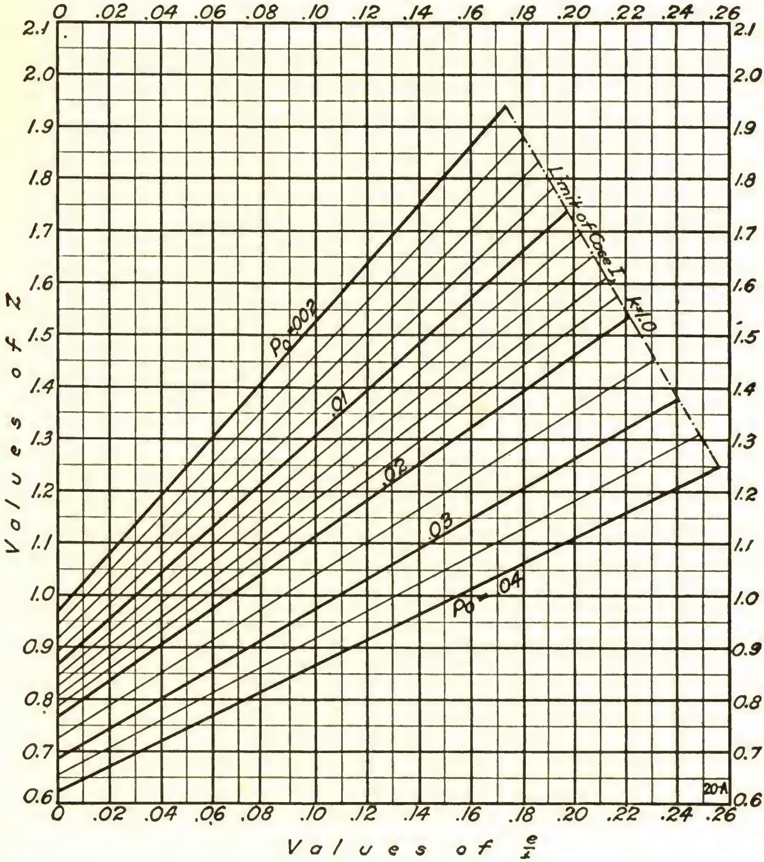
DIAGRAM 29.—CASE II—5000-LB. CONCRETE—$n = 6$



See instructions for use under Diagram 25.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

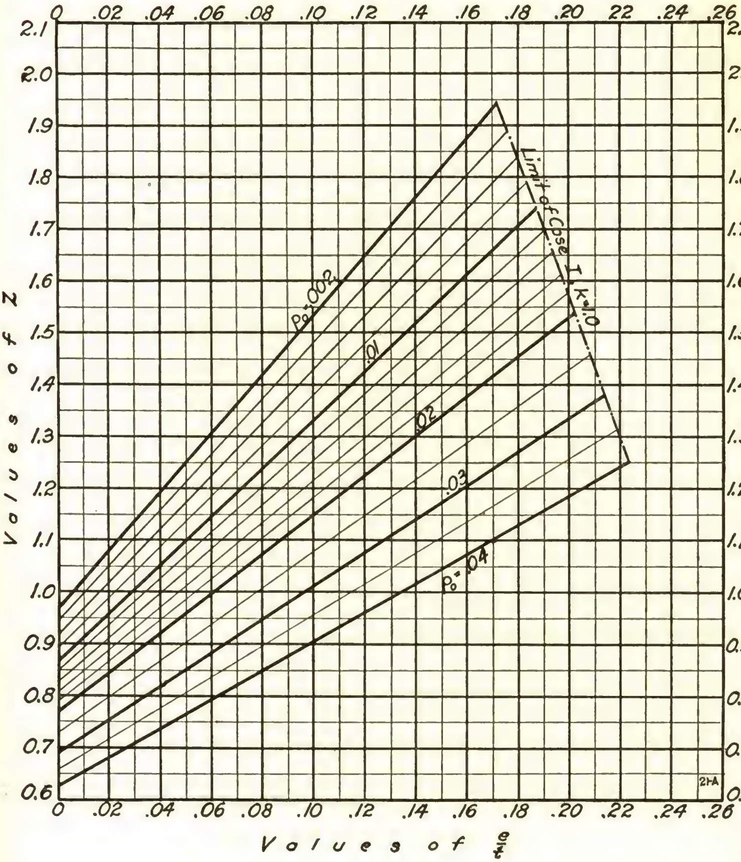
DIAGRAM 30.—CASE I—2000-LB. CONCRETE—n = 15—d' = 0.05t



INSTRUCTIONS FOR USE OF DIAGRAMS 30 TO 45.—Enter the diagram with the value of e/t and proceed vertically to an intersection with the sloping index line for an assumed value of p_0. From this intersection pass horizontally to the right or left marginal scale and read off the value of Z. Solve formula (110) for the stress in the concrete. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

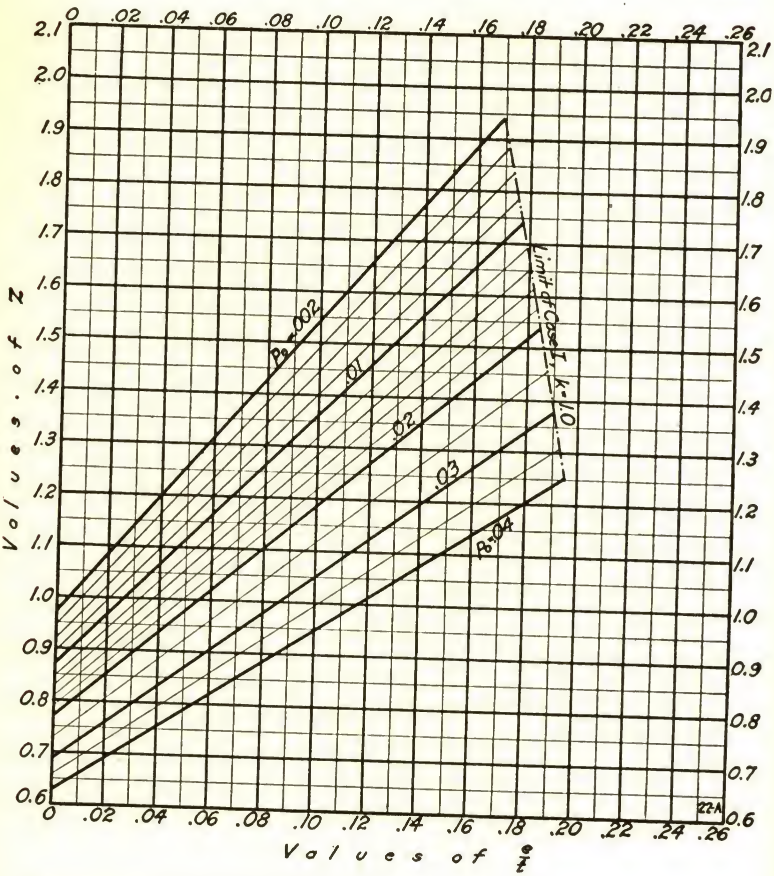
DIAGRAM 31.—CASE I—2000-LB. CONCRETE—$n = 15$—$d' = 0.1t$



See instructions for use under Diagram 30, page 66.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

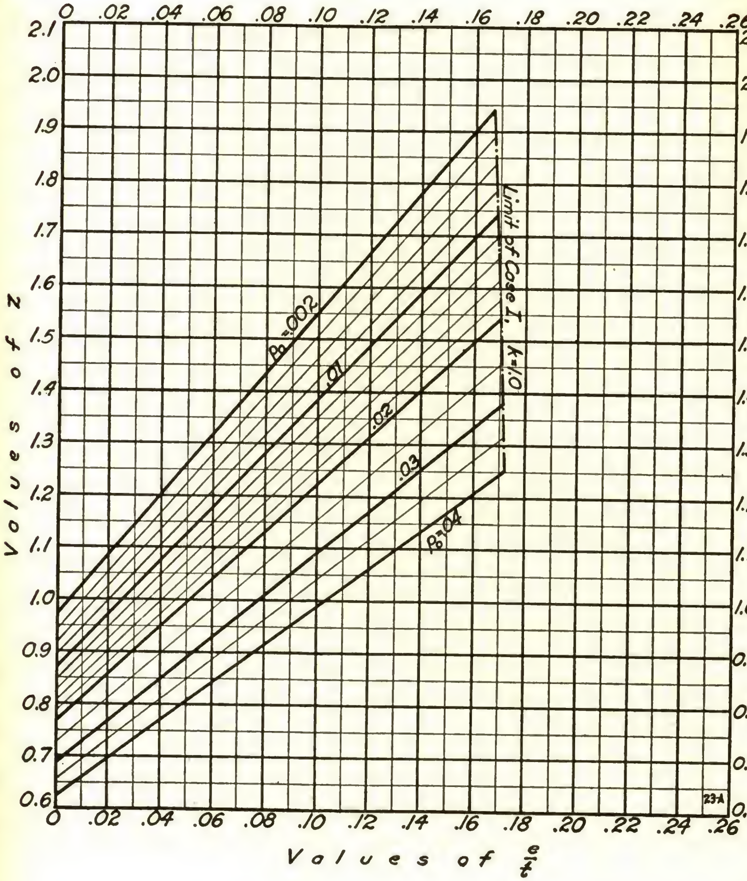
DIAGRAM 32.—CASE I—2000-LB. CONCRETE—$n = 15$—$d' = 0.15t$



See instructions for use under Diagram 30, page 66.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

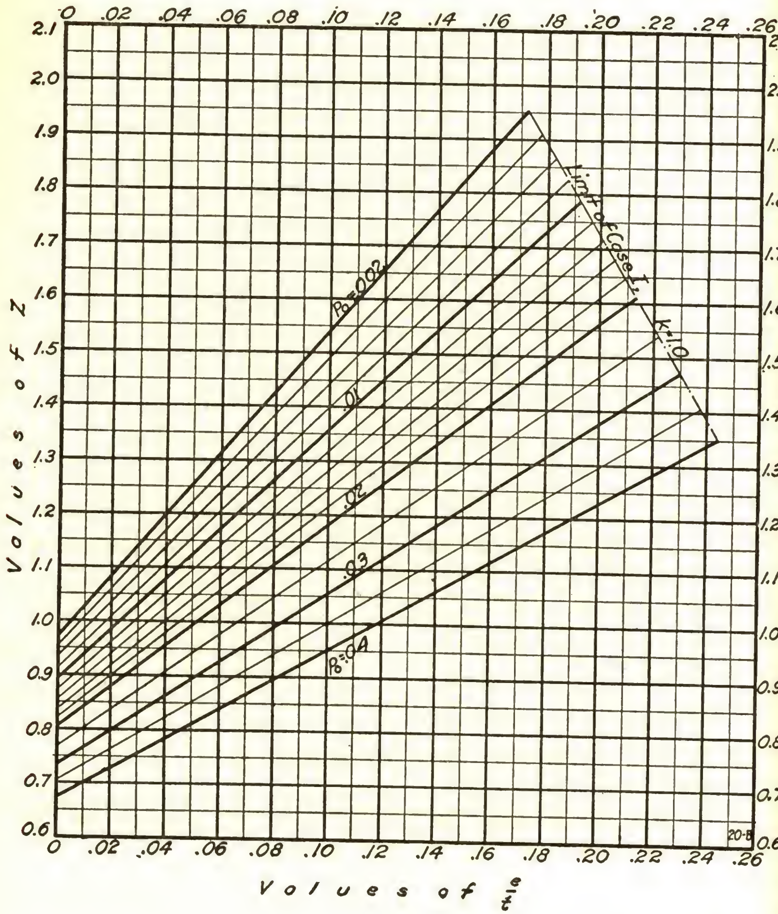
DIAGRAM 33.—CASE I—2000-LB. CONCRETE—$n = 15$—$d' = 0.2t$



See instructions for use under Diagram 30, page 66.

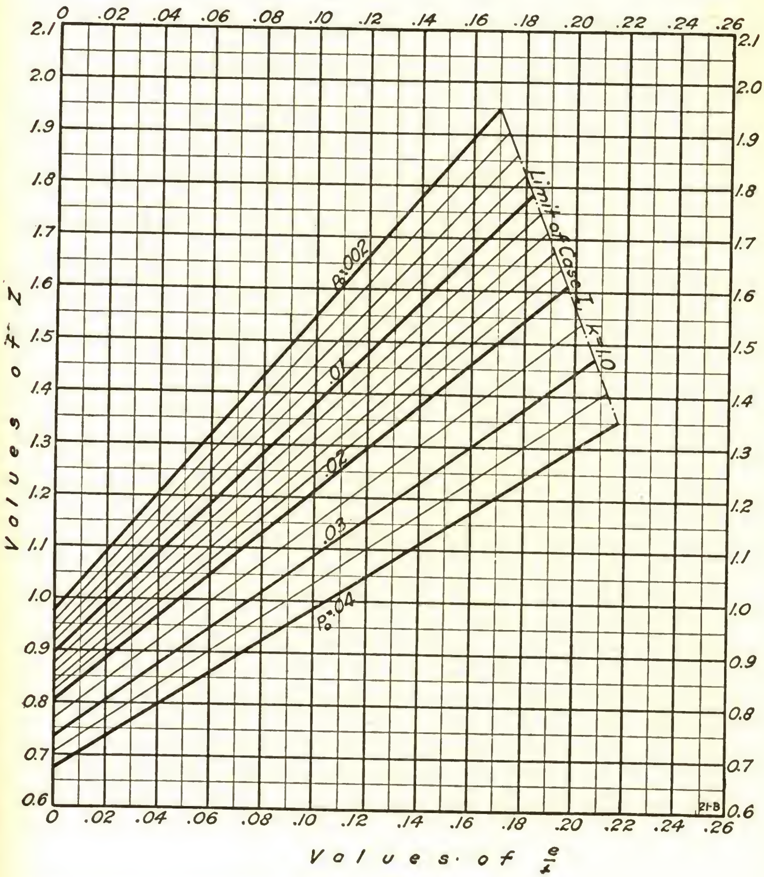
BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

DIAGRAM 34.—CASE I—2500-LB. CONCRETE—$n = 12$—$d' = 0.05t$



See instructions for use under Diagram 30, page 66.

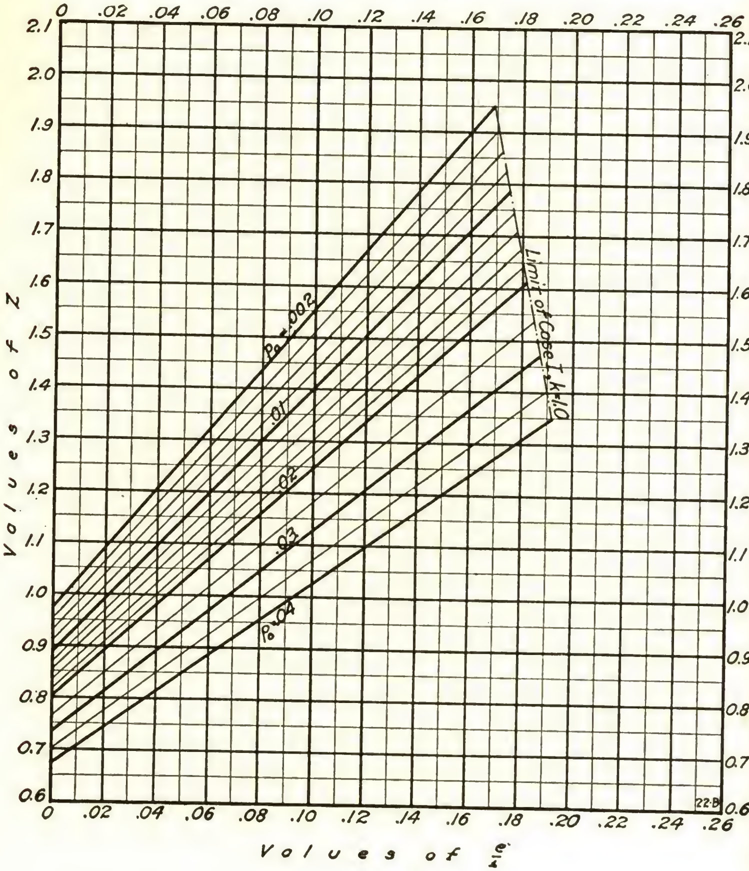
BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS
DIAGRAM 35.—CASE I—2500-LB. CONCRETE—$n = 12$—$d' = 0.1t$



See instructions for use under Diagram 30, page 66.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

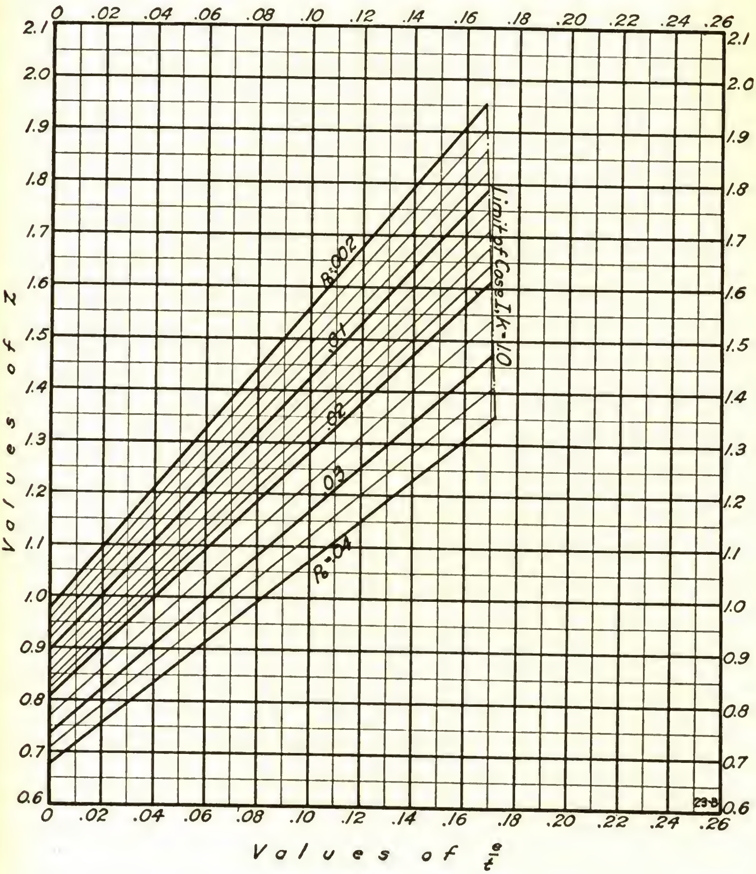
DIAGRAM 36.—CASE I—2500-LB. CONCRETE—$n = 12$—$d' = 0.15t$



See instructions for use under Diagram 30, page 66.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

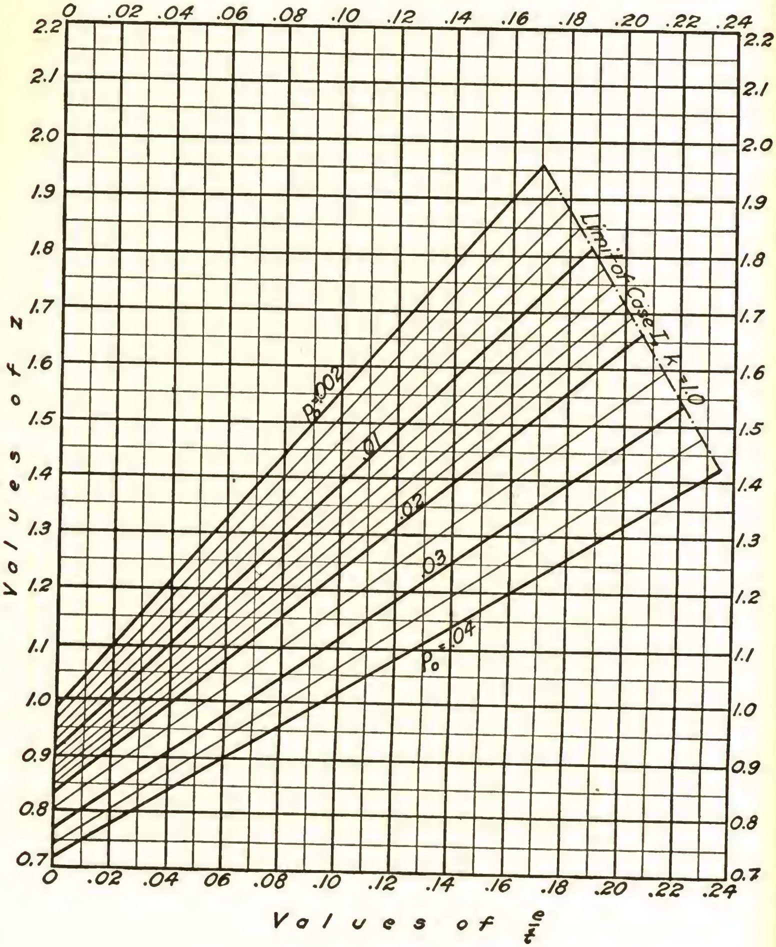
DIAGRAM 37.—CASE I—2500-LB. CONCRETE—$n = 12$—$d' = 0.2t$



See instructions for use under Diagram 30, page 66.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

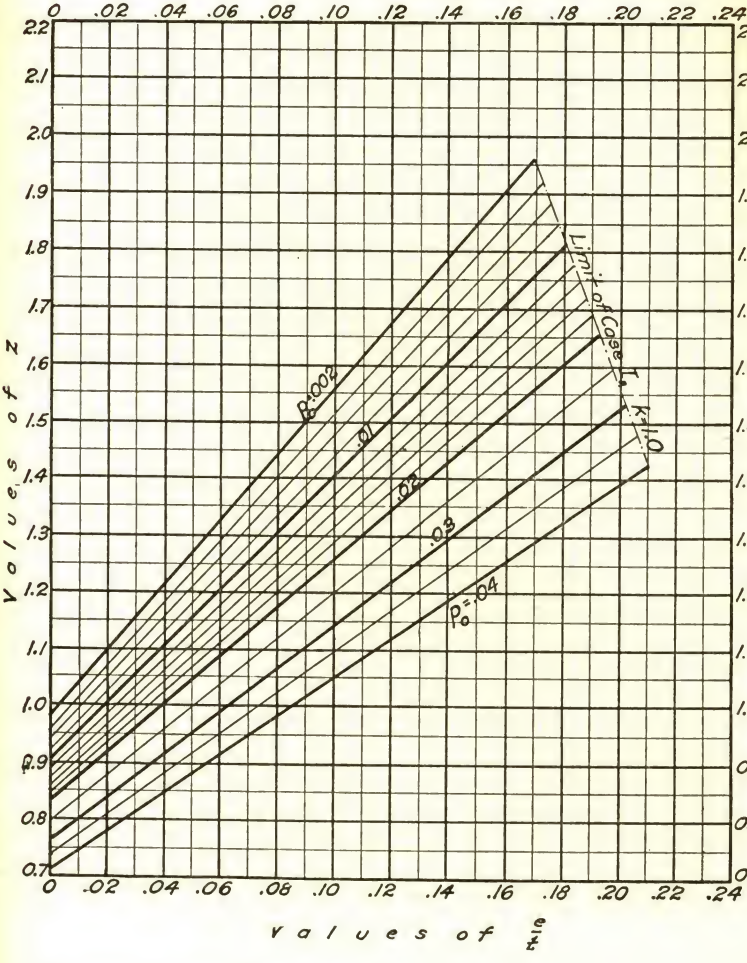
DIAGRAM 38.—CASE I—3000-LB. CONCRETE—$n = 10$ $d' = 0.05t$



See instructions for use under Diagram 30, page 66.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

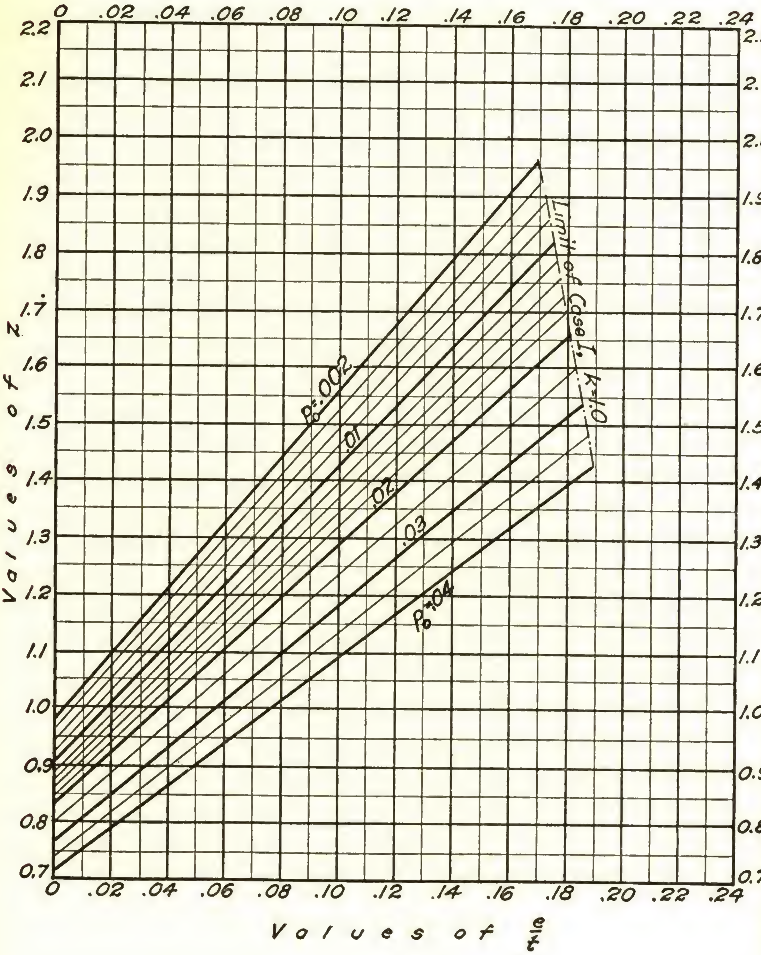
DIAGRAM 39.—CASE I—3000-LB. CONCRETE—$n = 10 \quad d' = 0.1t$



See instructions for use under Diagram 30, page 66.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

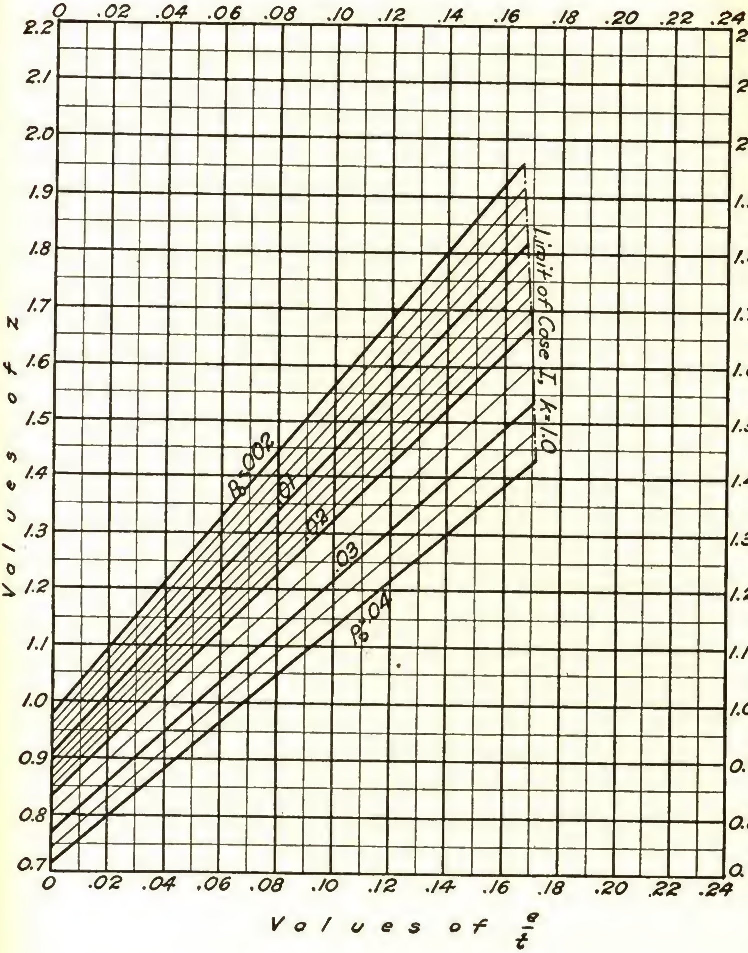
DIAGRAM 40.—CASE I—3000-LB. CONCRETE—$n = 10 \quad d' = 0.15t$



See instructions for use under Diagram 30, page 66.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

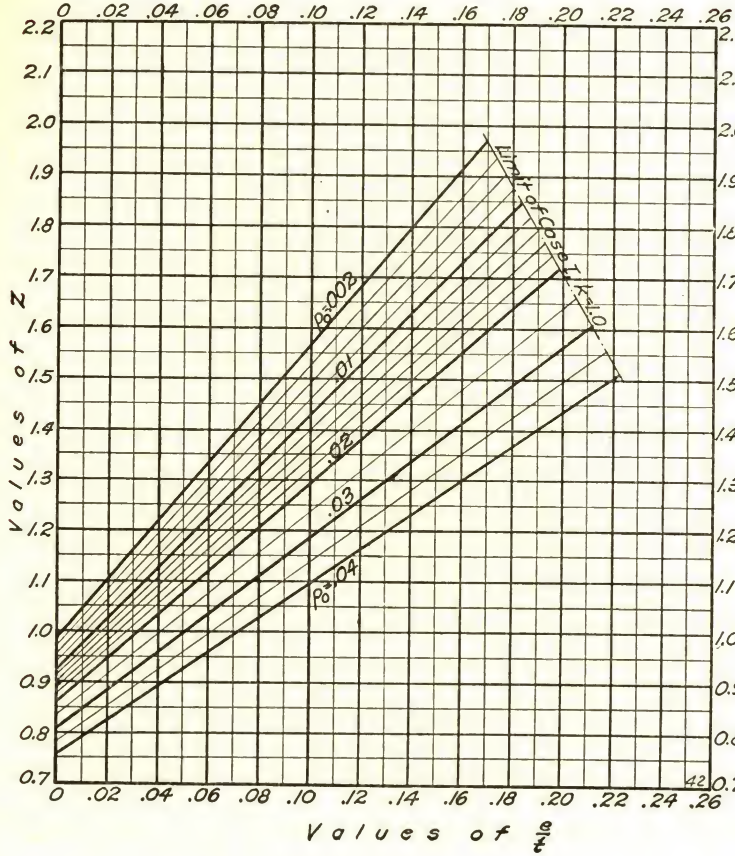
DIAGRAM 41.—CASE I—3000-LB. CONCRETE—$n = 10$ $d' = 0.2t$



See instructions for use under Diagram 30, page 66.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

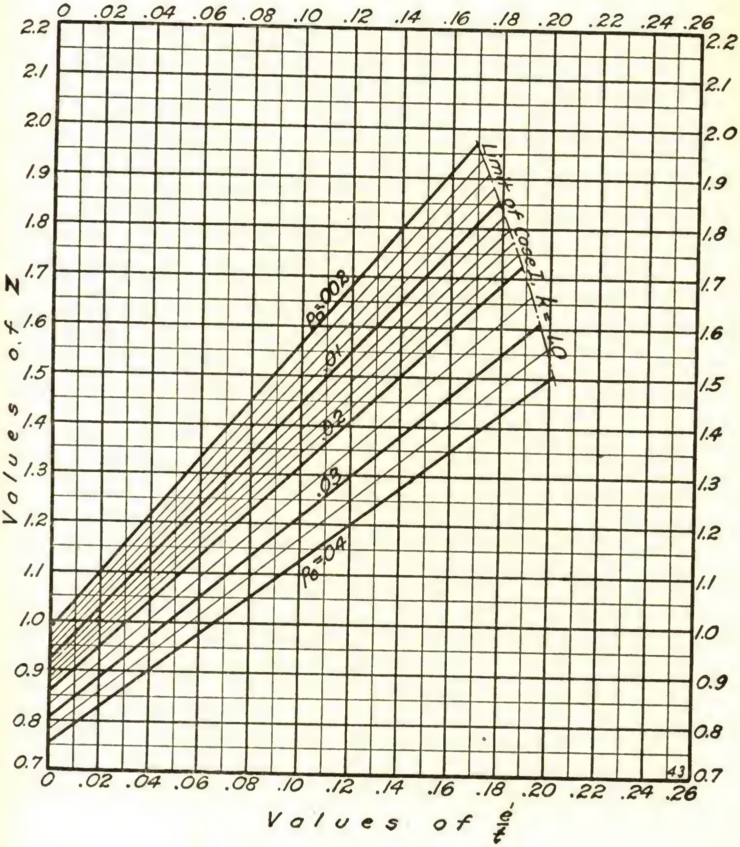
DIAGRAM 42.—CASE I—3750-LB. CONCRETE—n = 8—d' = 0.05t



See instructions for use under Diagram 30, page 66.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

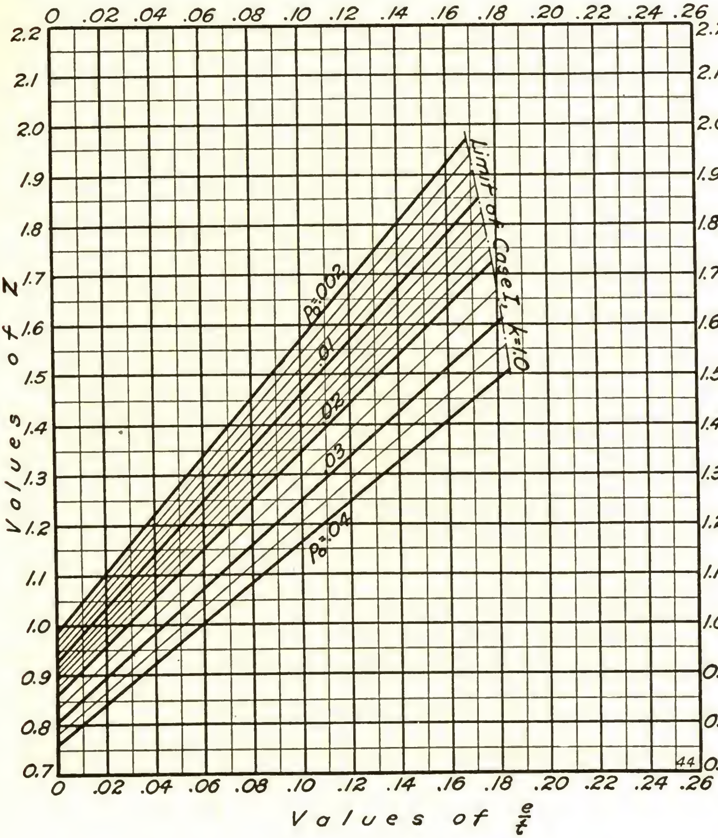
DIAGRAM 43.—CASE I—3750-LB. CONCRETE—$n = 8$—$d' = 0.1t$



See instructions for use under Diagram 30, page 66.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

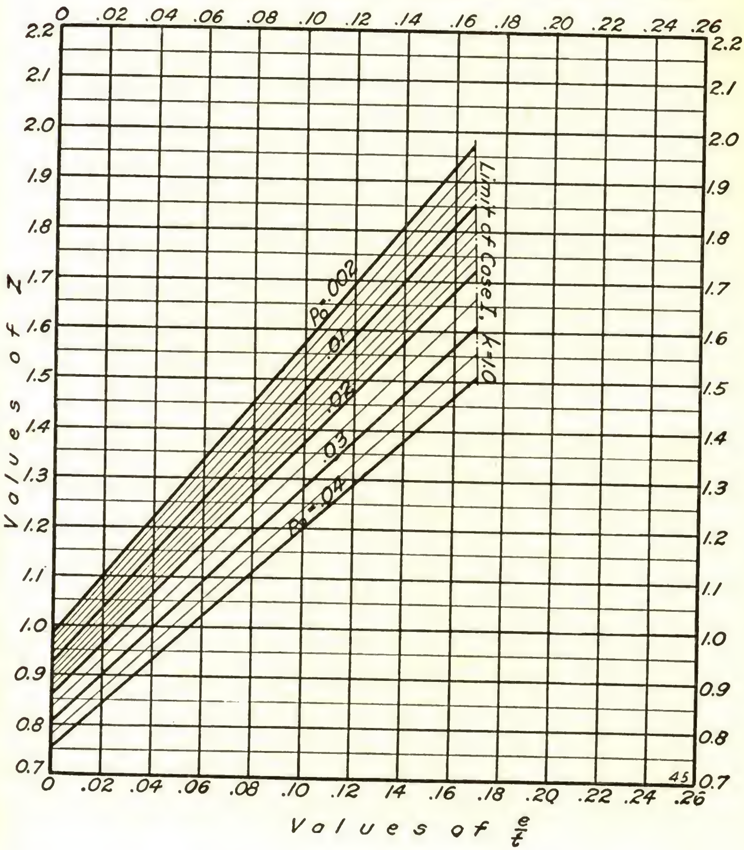
DIAGRAM 44.—CASE I—3750-LB. CONCRETE—$n = 8$—$d' = 0.15t$



See instructions for use under Diagram 30, page 66.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

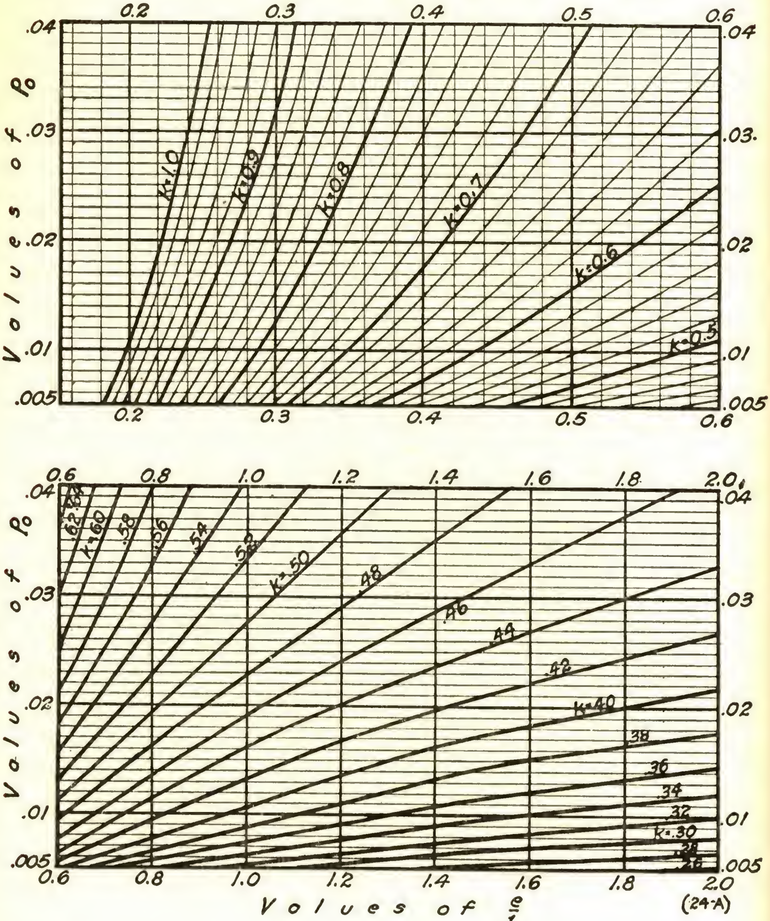
DIAGRAM 45.—CASE I—3750-LB. CONCRETE—$n = 8$—$d' = 0.2t$



See instructions for use under Diagram 30, page 66.

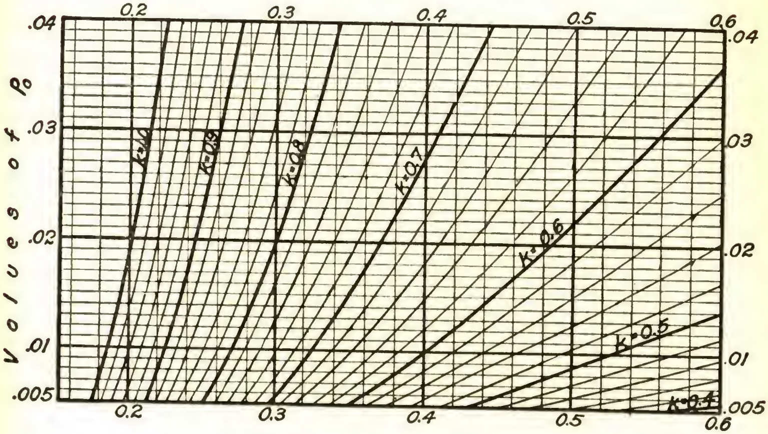
BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

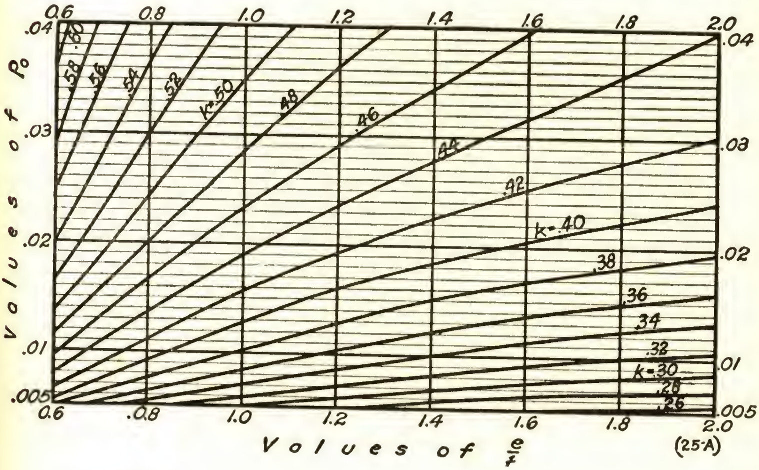
DIAGRAM 46.—CASE II—2000-LB. CONCRETE—n = 15—d' = 0.05t



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersect on with a horizontal line drawn through an assumed value of po on the left or right marginal scale. Read off on the inclined scales the value of k. With this value of k enter Diagram 50. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS
DIAGRAM 47.—CASE II—2000-LB. CONCRETE—$n = 15$—$d' = 0.1t$

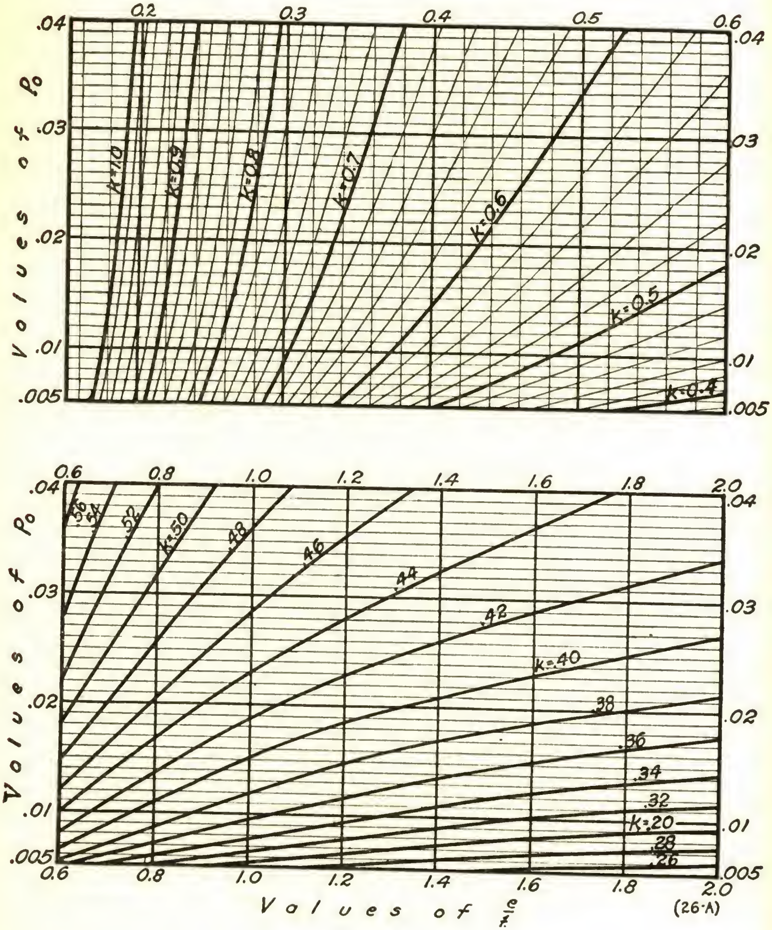




INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_o on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 50. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

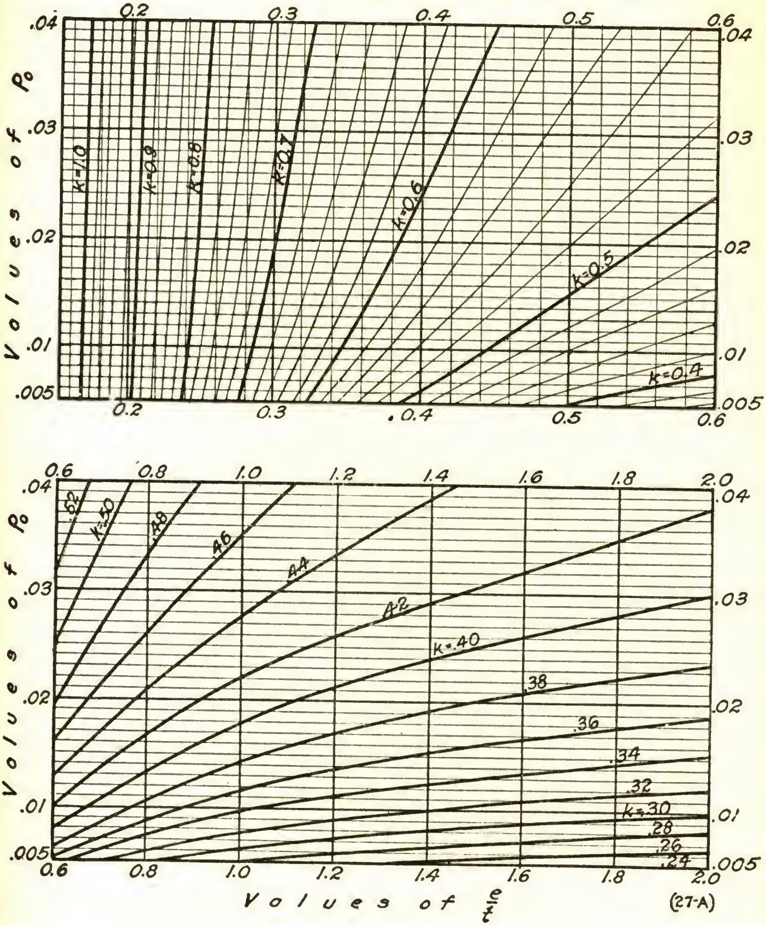
DIAGRAM 48.—CASE II—2000-LB. CONCRETE—$n = 15$—$d' = 0.15t$



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_o on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 50. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

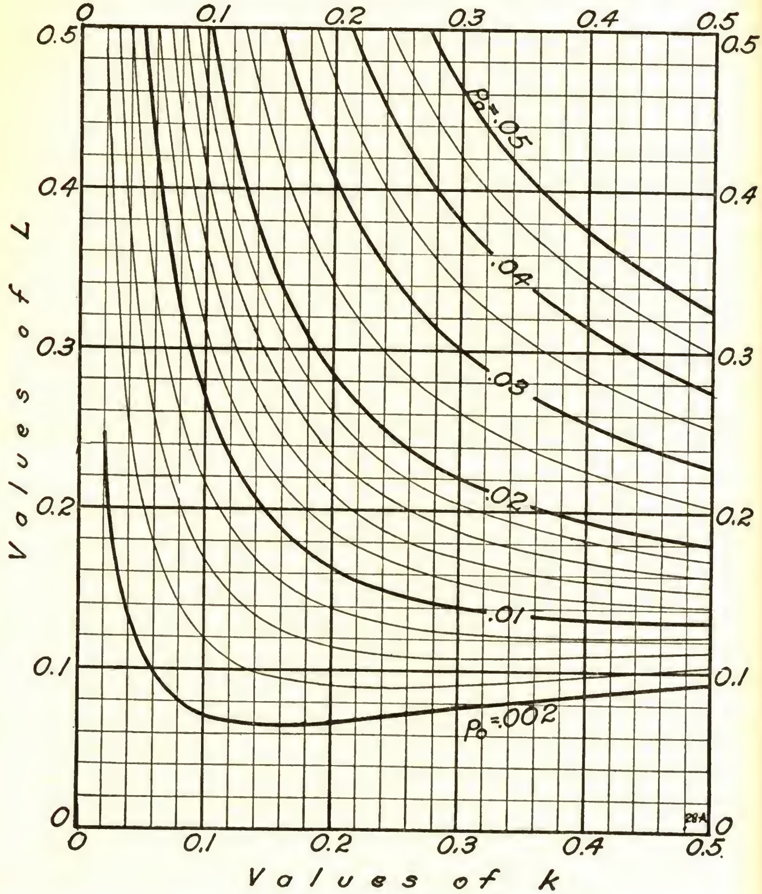
DIAGRAM 49.—CASE II—2000-LB. CONCRETE—$n = 15$—$d' = 0.2t$



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_o on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 50. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

DIAGRAM 50.—(PART ONE)—CASE II—2000-LB. CONCRETE—$n = 15$

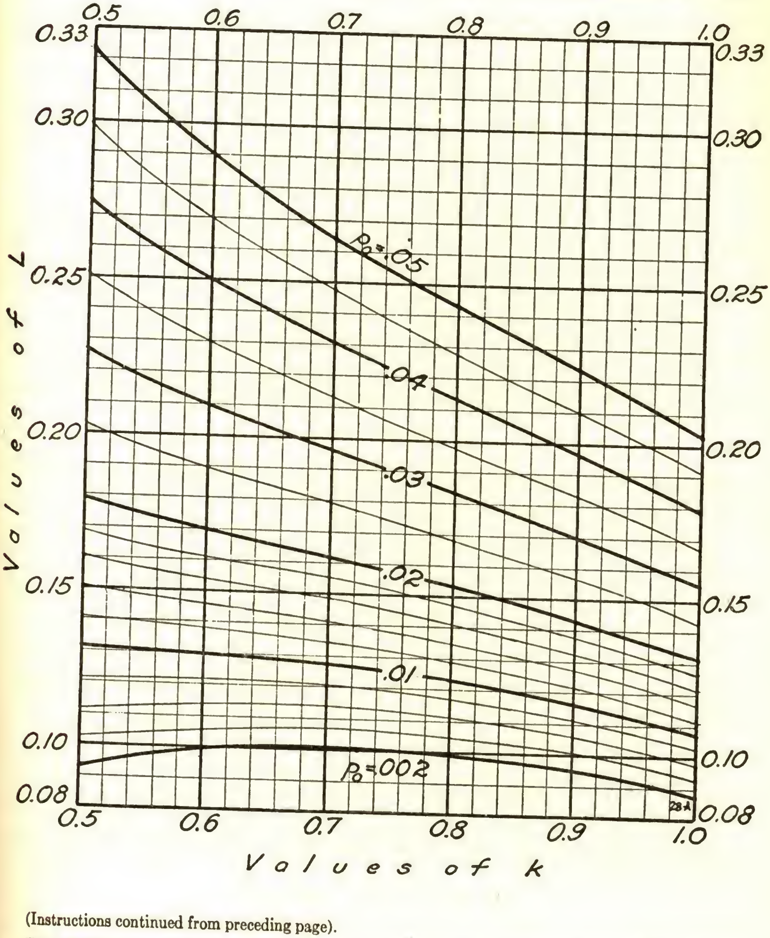


INSTRUCTIONS FOR USE.—The value of k must first be obtained from Diagrams 46 (or 47, 48 or 49, according to value of d'/t in the member being designed). The value of po used in those diagrams must be modified as follows:

po used in Diagram 46 must be divided by 0.79 ($d' = 0.05t$)
po used in Diagram 47 used without modification ($d' = 0.1t$)
po used in Diagram 48 must be divided by 1.306 ($d' = 0.15t$)
po used in Diagram 49 must be divided by 1.78 ($d' = 0.2t$)

(Instructions continued under Part Two).

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS
DIAGRAM 50.—(PART TWO)—CASE II—2000-LB. CONCRETE—$n = 15$

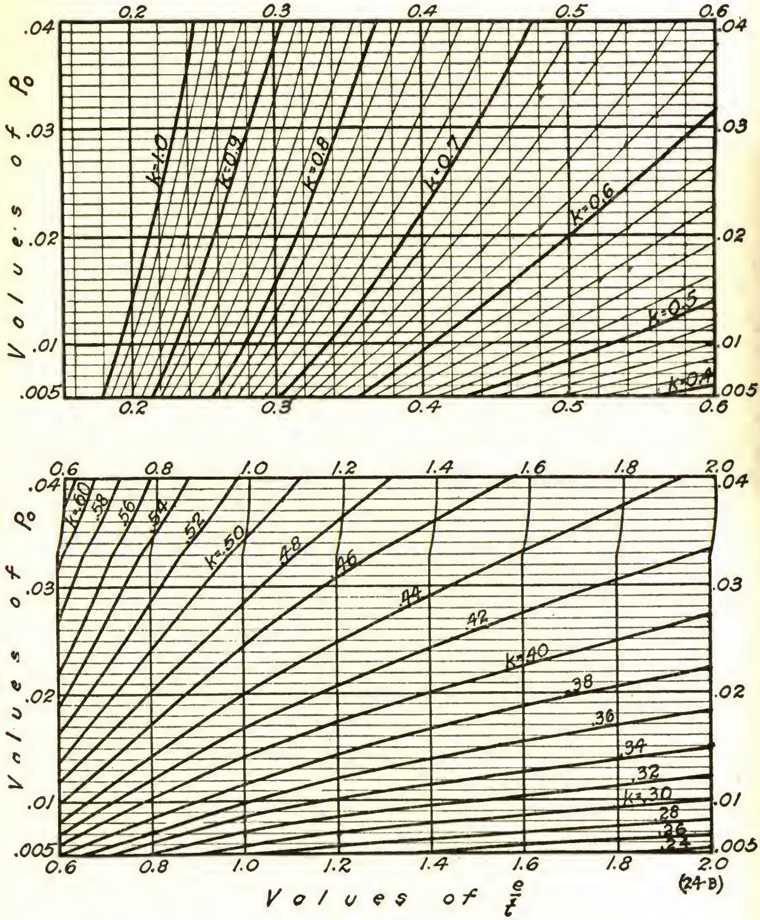


(Instructions continued from preceding page).

Enter the diagram with the value of k and proceed vertically to an intersection with the sloping index line corresponding to the modified value of p_O. From this intersection pass horizontally to the left or right marginal scale and read off the value of L. Solve formulas (31) and (32) for the stresses in the concrete and reinforcing steel. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

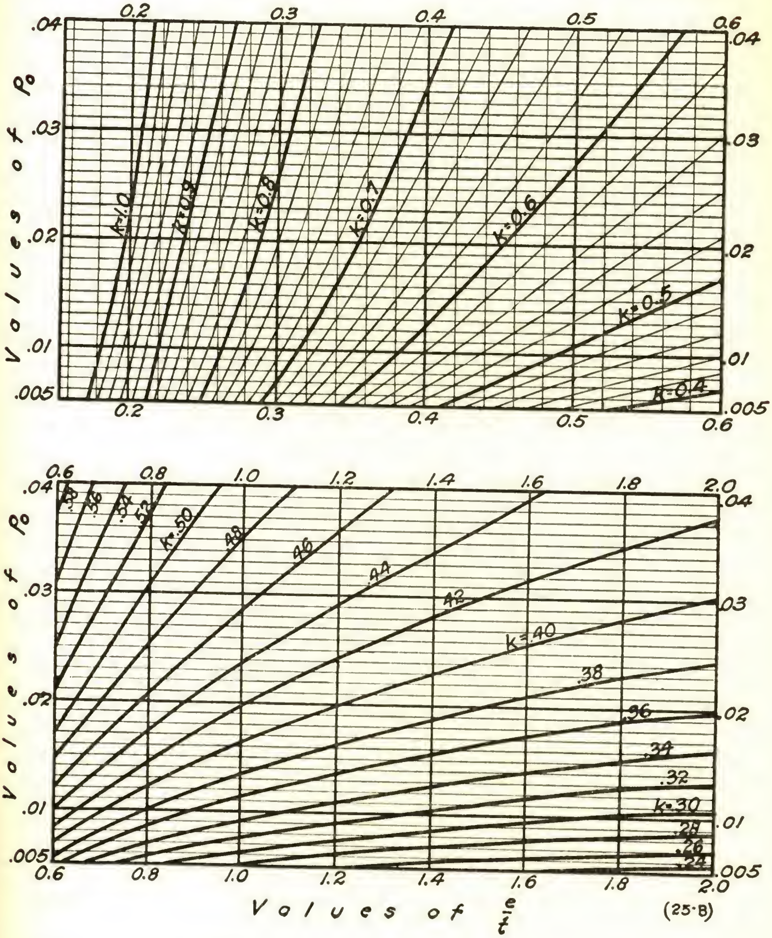
DIAGRAM 51.—CASE II—2500-LB. CONCRETE—$n = 12$—$d' = 0.05t$



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_o on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 55. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

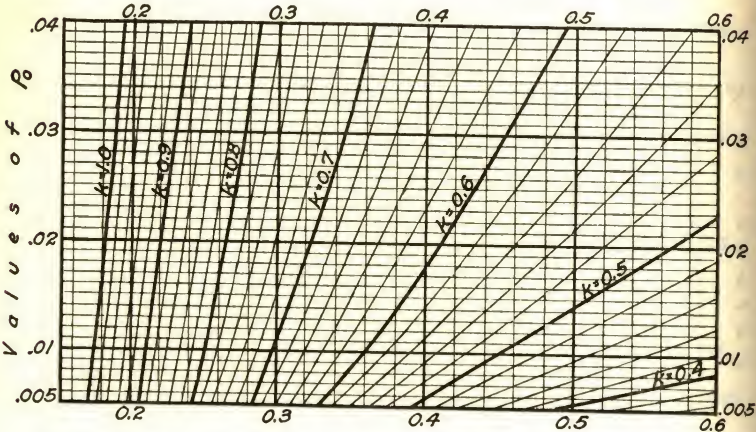
DIAGRAM 52.—CASE II—2500-LB. CONCRETE—$n = 12$—$d' = 0.1t$

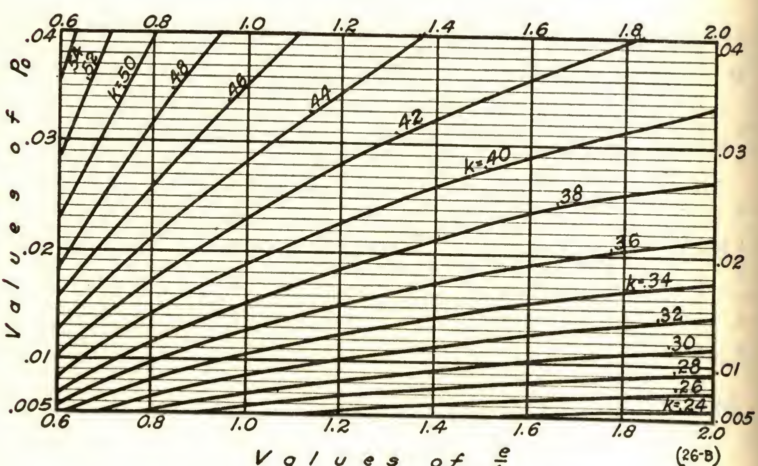


INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_0 on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 55. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

DIAGRAM 53.—CASE II—2500-LB. CONCRETE—$n = 12$—$d' = 0.15t$

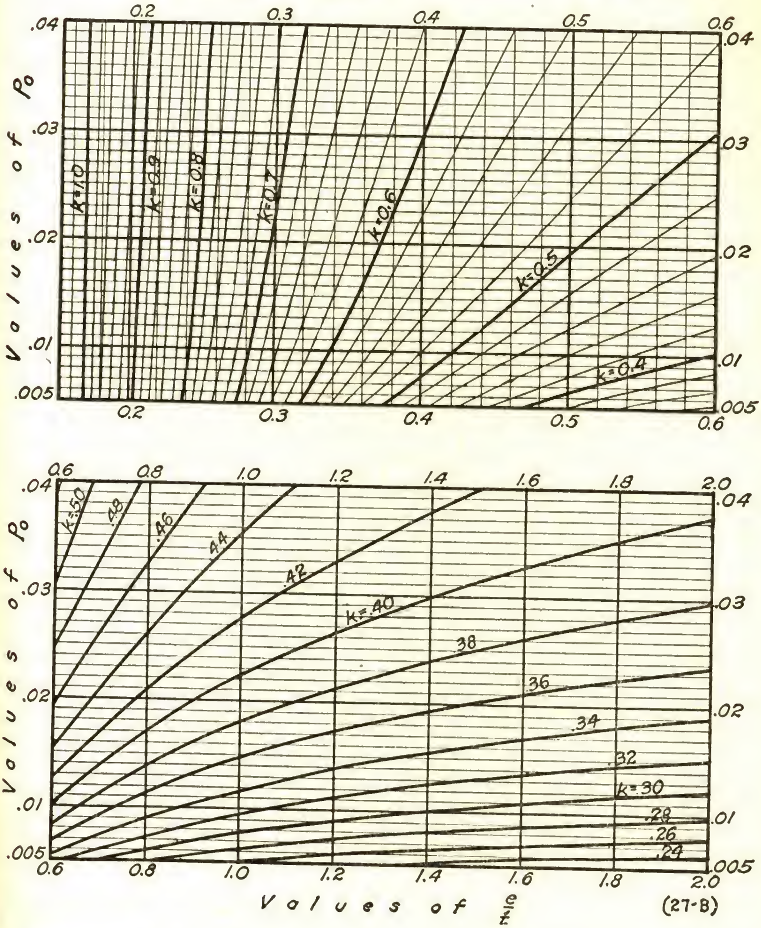




INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_o on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 55. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

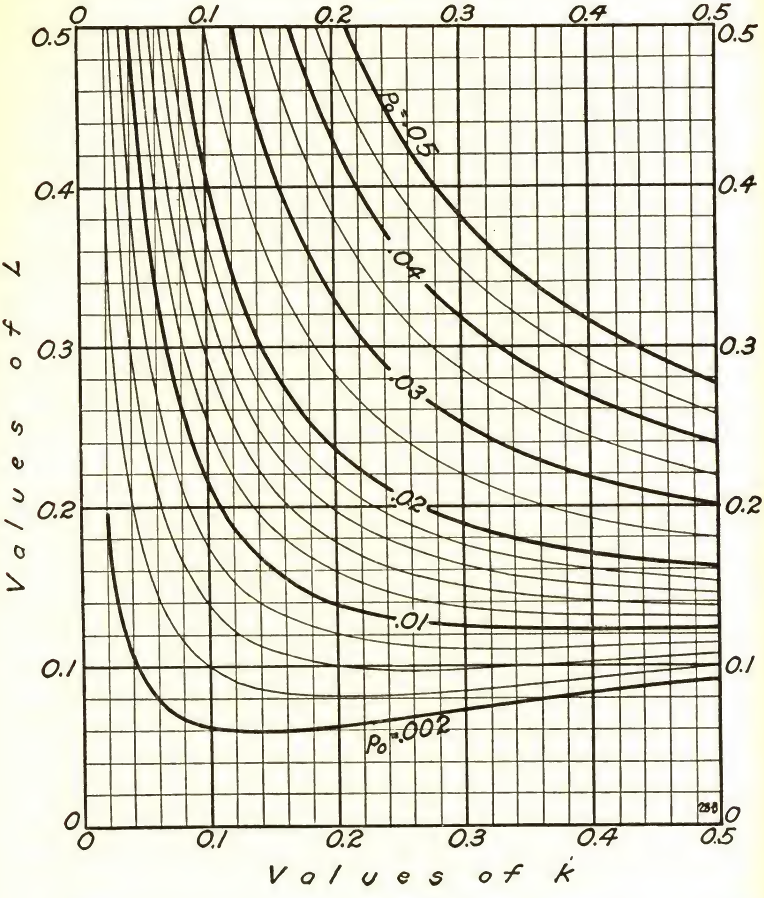
DIAGRAM 54.—CASE II—2500-LB. CONCRETE—$n = 12$—$d' = 0.2t$



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_o on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 55. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

DIAGRAM 55.—(PART ONE)—CASE II—2500-LB. CONCRETE—n = 12



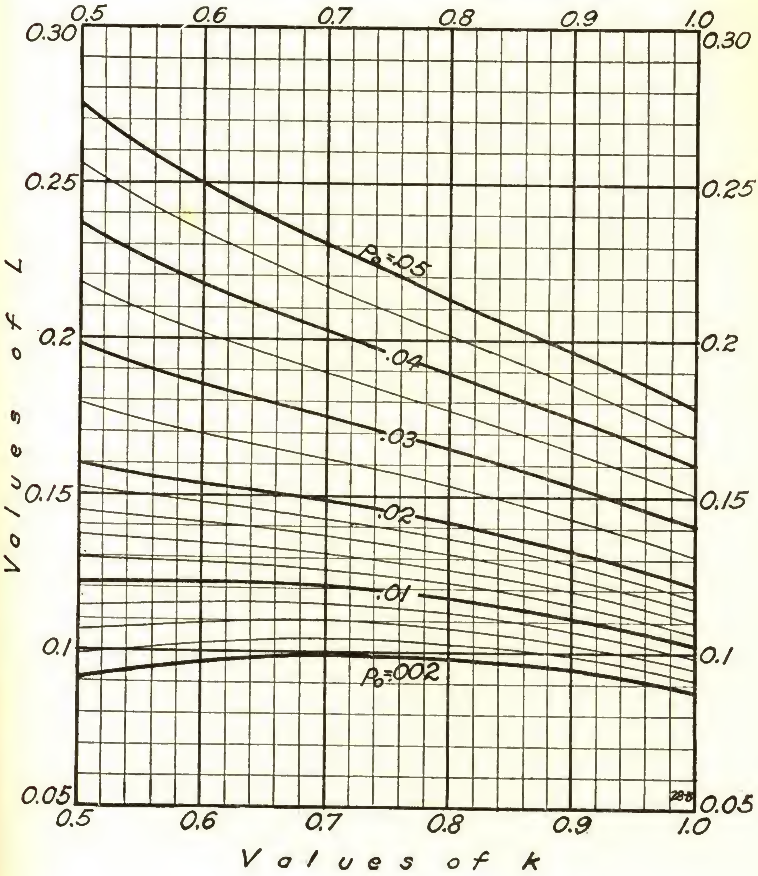
INSTRUCTIONS FOR USE.—The value of k must first be obtained from Diagrams 51 (or 52, 53, or 54, according to value of d'/t in the member being designed). The value of p_o used in those diagrams must be modified as follows:

p_o used in Diagram 51 must be divided by 0.79 ($d' = 0.05t$)
p_o used in Diagram 52 used without modification ($d' = 0.1t$)
p_o used in Diagram 53 must be divided by 1.306 ($d' = 0.15t$)
p_o used in Diagram 54 must be divided by 1.78 ($d' = 0.2t$)

(Instructions continued under Part Two)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

DIAGRAM 55.—(PART TWO)—CASE II—2500-LB. CONCRETE—$n = 12$

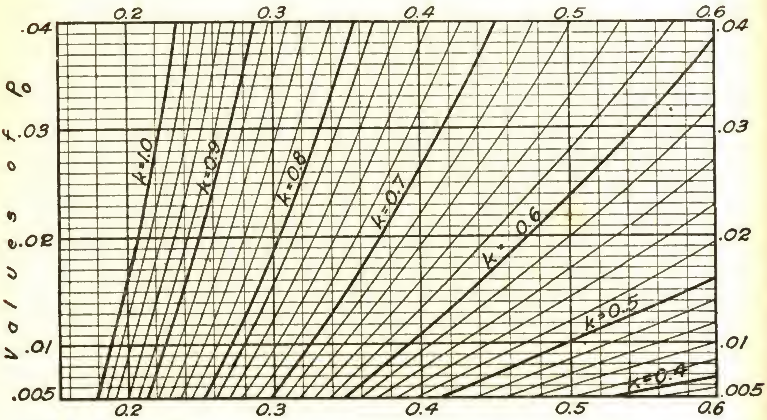


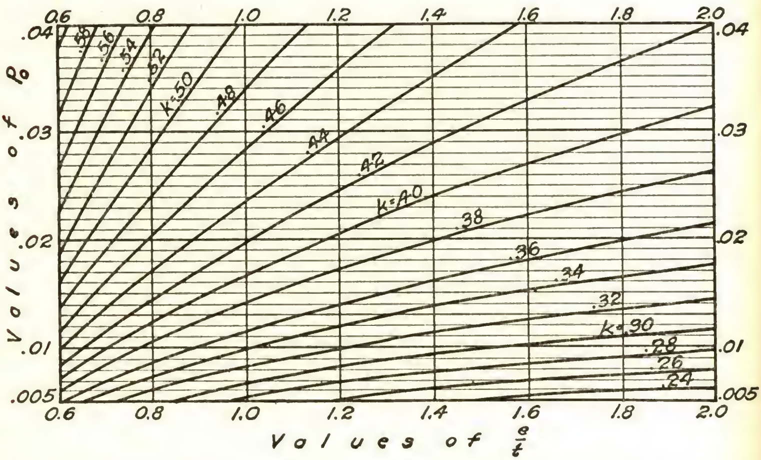
(Instructions continued from preceding page)

Enter the diagram with the value of k and proceed vertically to an intersection with the sloping index line corresponding to the modified value of p_o. From this intersection pass horizontally to the left or right marginal scale and read off the value of L. Solve formulas (31) and (32) for the stresses in the concrete and reinforcing steel. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

DIAGRAM 56.—CASE II—3000-LB. CONCRETE—$n = 10 \quad d' = 0.05t$

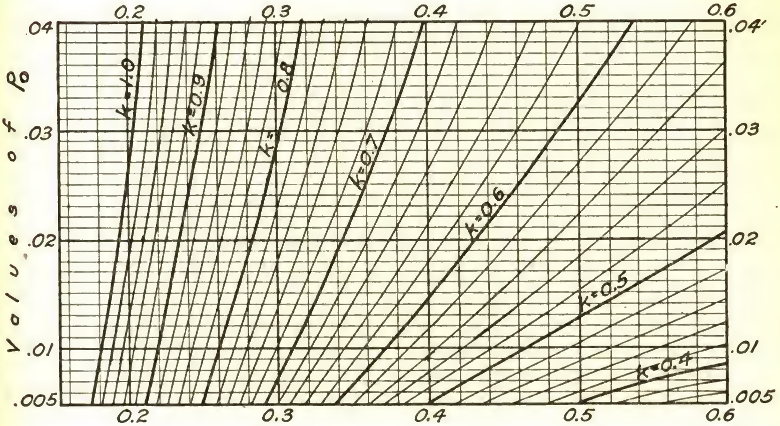


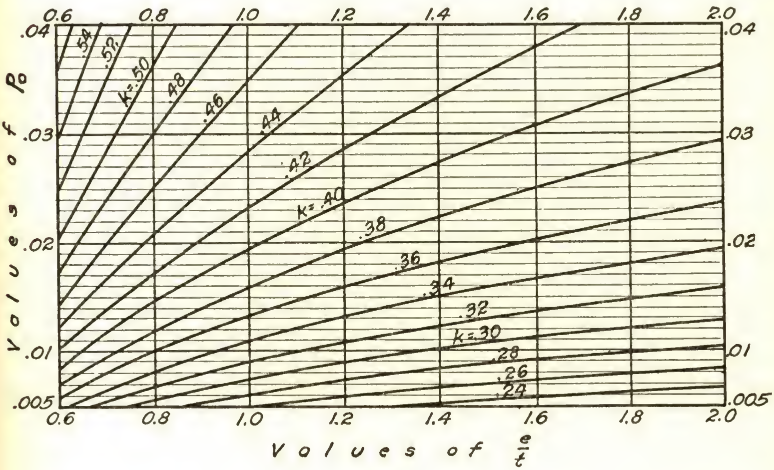


INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_o on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 60. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

DIAGRAM 57.—CASE II—3000-LB. CONCRETE—$n = 10$ $d' = 0.1t$

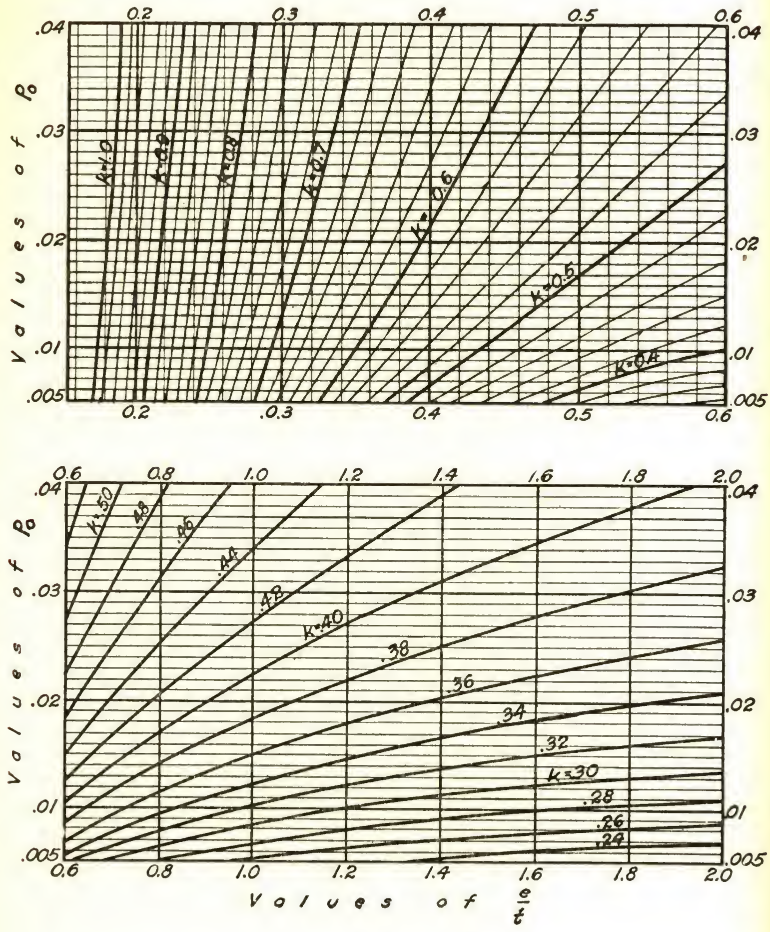




INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_0 on the left or right marginal scale. Read off on the inclined scales the value of k. With this value of k enter Diagram 60. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

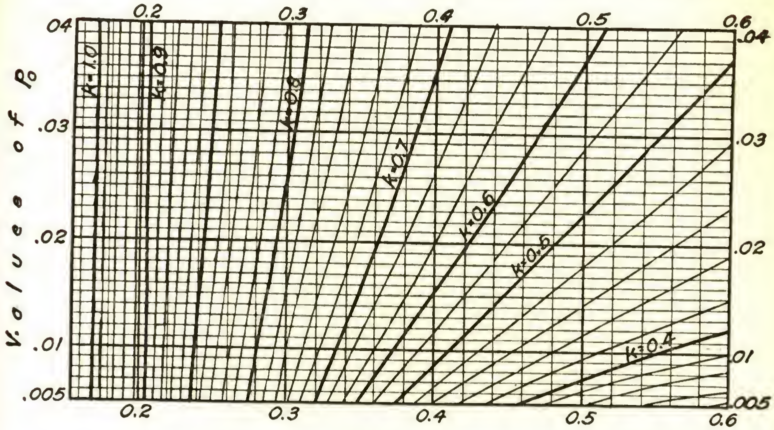
DIAGRAM 58.—CASE II—3000-LB. CONCRETE—$n = 10$ $d' = 0.15t$

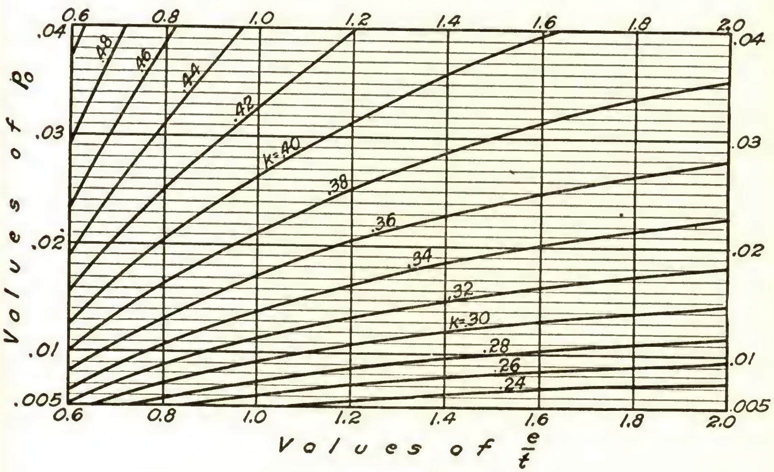


INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_0 on the left or right marginal scale. Read off on the inclined scales the value of k. With this value of k enter Diagram 60. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

DIAGRAM 59.—CASE II—3000-LB. CONCRETE—$n = 10$ $d' = 0.2t$

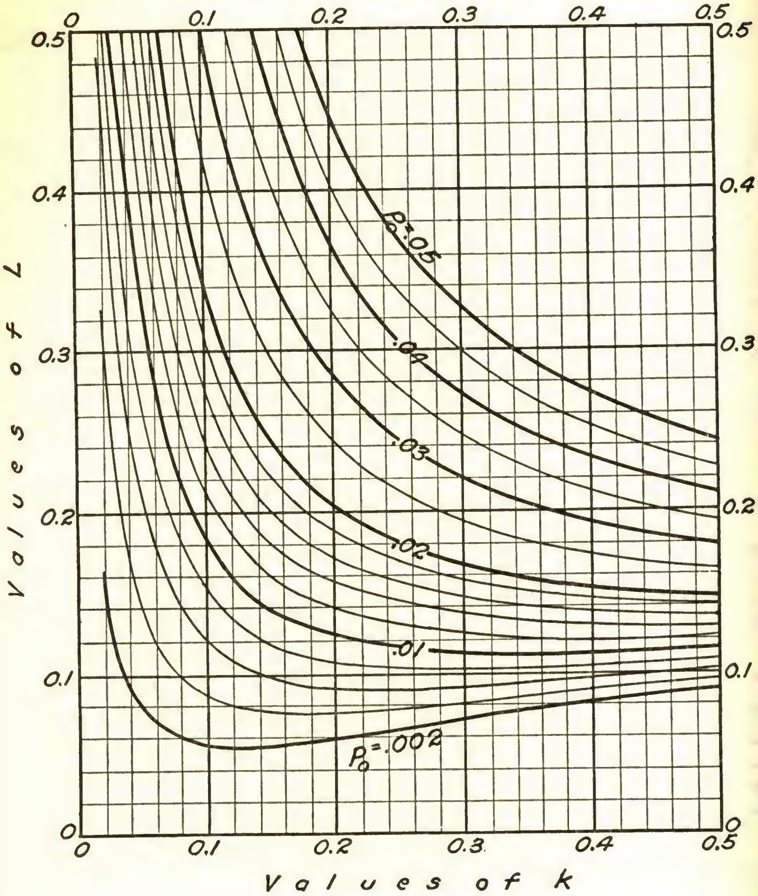




INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_0 on the left or right marginal scale. Read off on the inclined scales the value of k. With this value of k enter Diagram 60. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

DIAGRAM 60.—(PART ONE)—CASE II—3000-LB. CONCRETE—n = 10



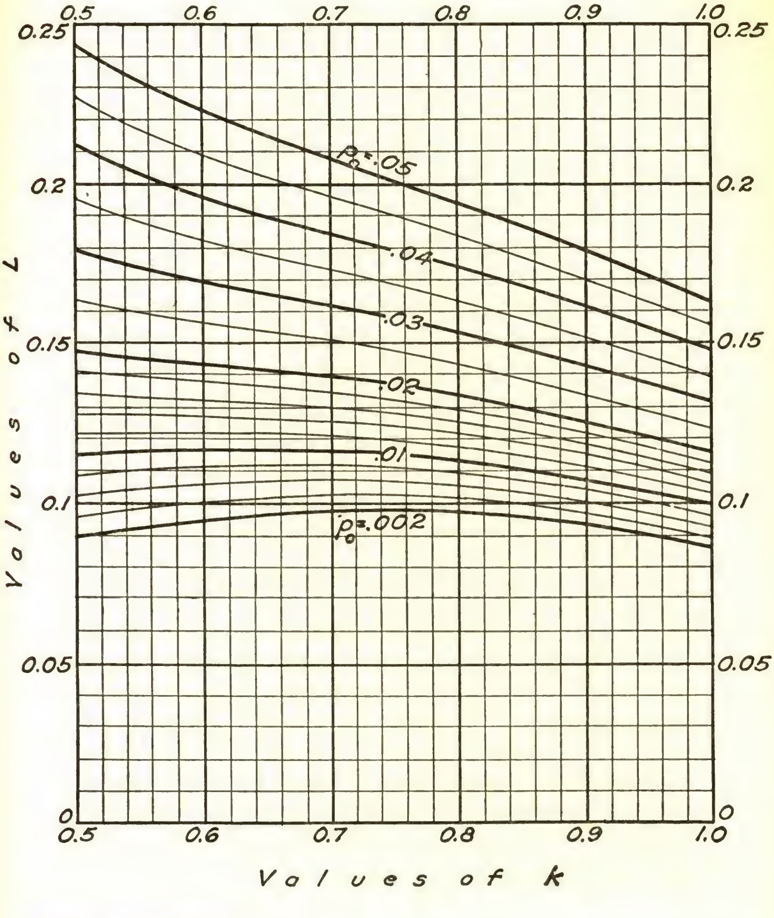
INSTRUCTIONS FOR USE.—The value of k must first be obtained from Diagrams 56 (or 57, 58 or 59, according to value of d'/t in the member being designed). The value of p_o used in those diagrams must be modified as follows:

p_o used in Diagram 56 must be divided by 0.79 ($d' = 0.05t$).
p_o used in Diagram 57 used without modification ($d' = 0.1t$).
p_o used in Diagram 58 must be divided by 1.306 ($d' = 0.15t$).
p_o used in Diagram 59 must be divided by 1.78 ($d' = 0.2t$).

(Instructions continued under Part Two.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

DIAGRAM 60.—(PART TWO)—CASE II—3000-LB. CONCRETE—$n = 10$

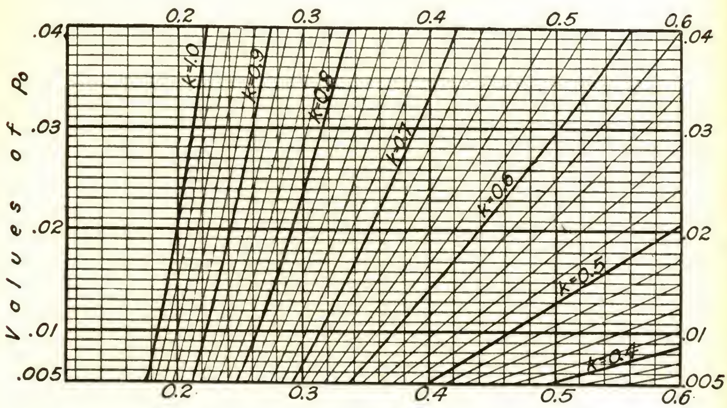


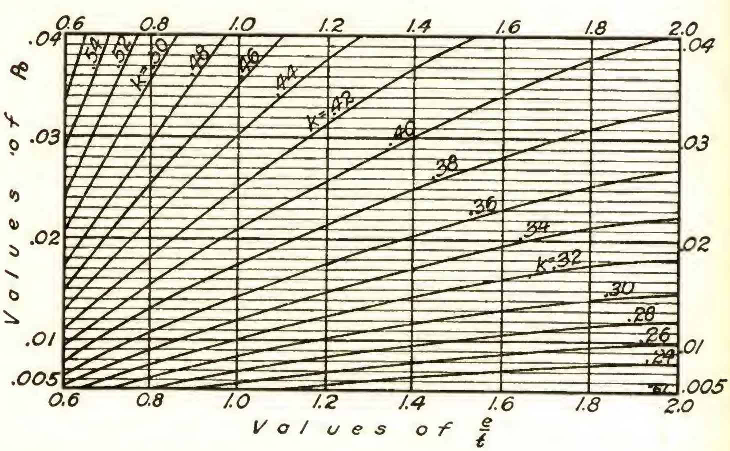
(Instructions continued from preceding page.)

Enter the diagram with the value of k and proceed vertically to an intersection with the sloping index line corresponding to the modified value of p_o. From this intersection pass horizontally to the left or right marginal scale and read off the value of L. Solve formulas (31) and (32) for the stresses in the concrete and reinforcing steel. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

DIAGRAM 61.—CASE II—3750-LB. CONCRETE—$n = 8$—$d' = 0.05t$

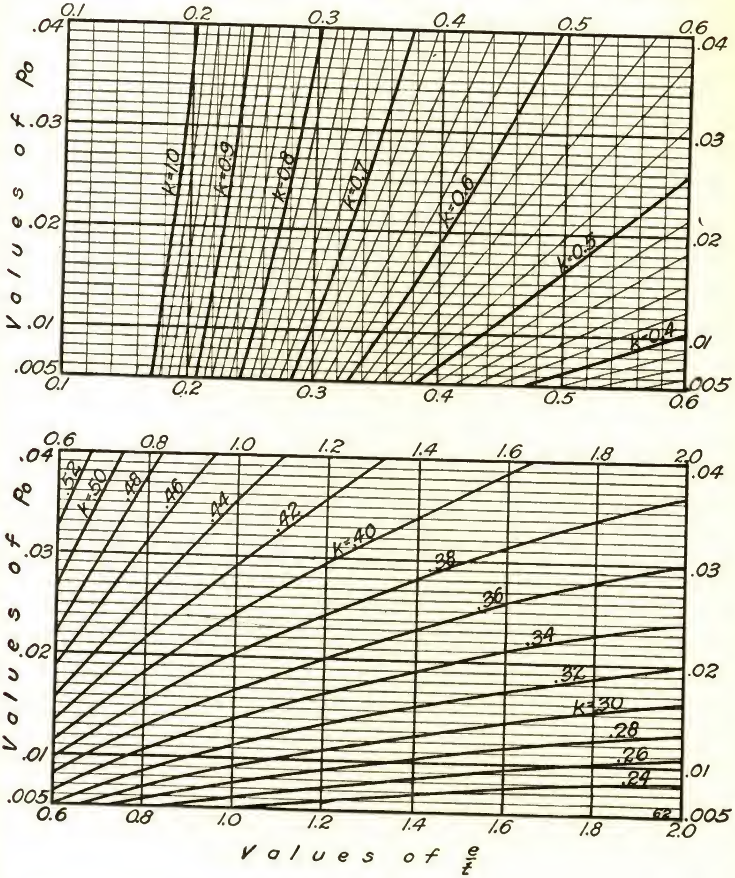




INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_0 on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 65. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

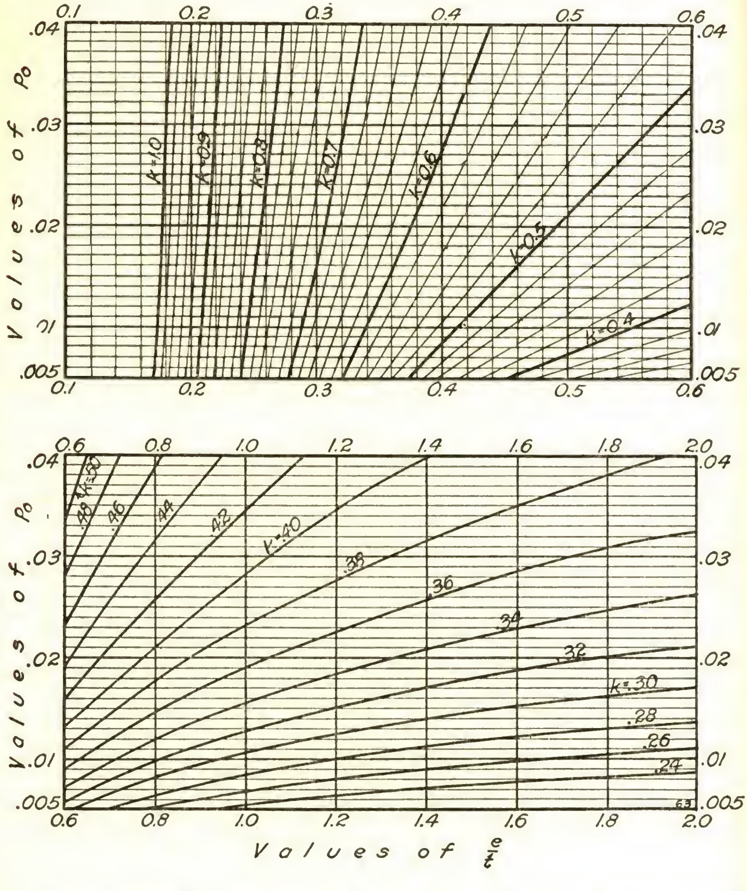
DIAGRAM 62.—CASE II—3750-LB. CONCRETE—$n = 8$—$d' = 0.1t$



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_o on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 65. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

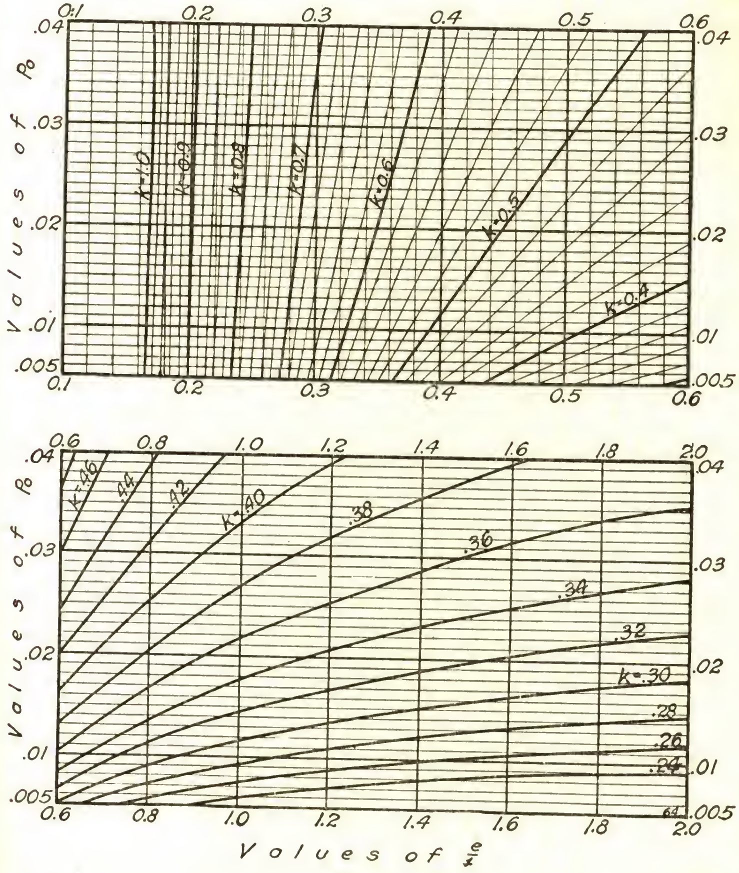
DIAGRAM 63.—CASE II—3750-LB. CONCRETE—$n = 8$—$d' = 0.15t$



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_o on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 65. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

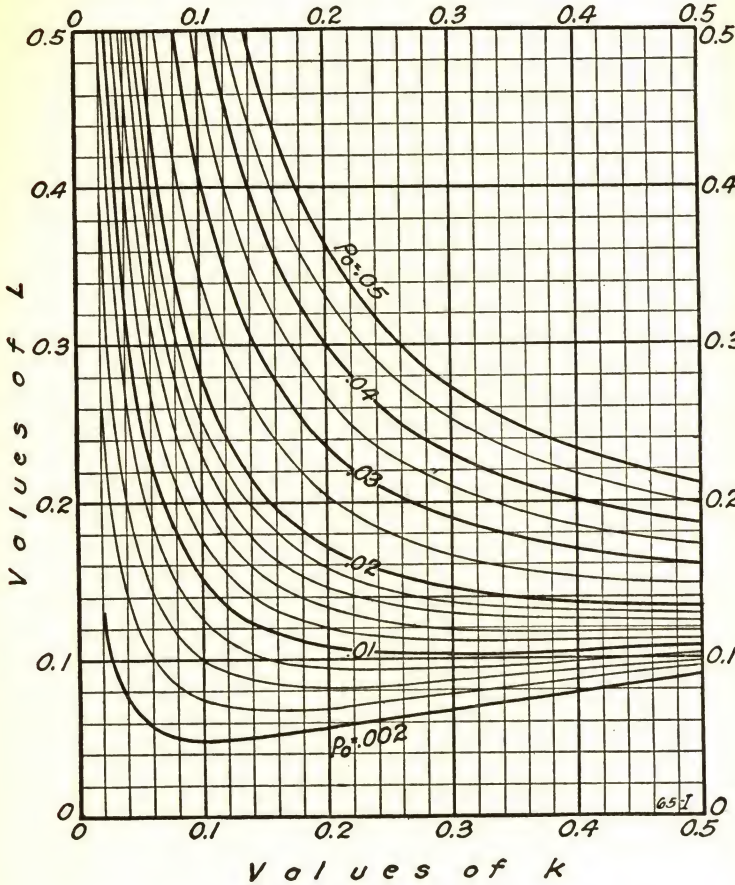
DIAGRAM 64.—CASE II—3750-LB. CONCRETE—$n = 8$—$d' = 0.2t$



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_o on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 65. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

DIAGRAM 65.—(PART ONE)—CASE II—3750-LB. CONCRETE—$n = 8$



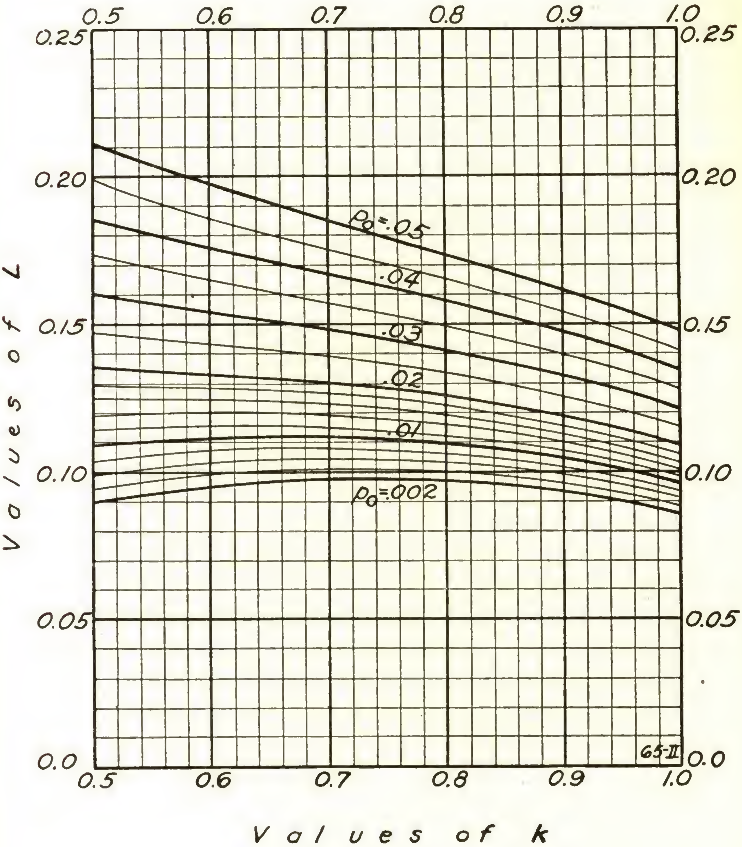
INSTRUCTIONS FOR USE.—The value of k must first be obtained from Diagrams 61 (or 62, 63 or 64, according to value of d'/t in the member being designed). The value of po used in those diagrams must be modified as follows:

po used in Diagram 61 must be divided by 0.79 ($d' = 0.05t$).
po used in Diagram 62 used without modification ($d' = 0.1t$)
po used in Diagram 63 must be divided by 1.306 ($d' = 0.15t$)
po used in Diagram 64 must be divided by 1.78 ($d' = 0.2t$)

(Instructions continued under Part Two.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

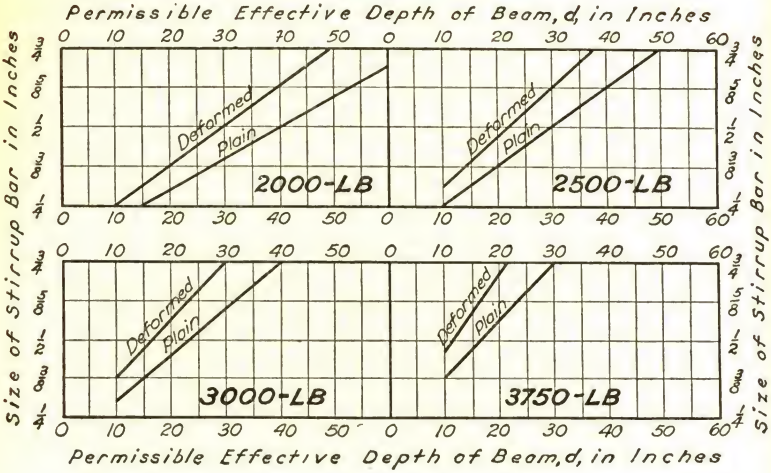
DIAGRAM 65.—(PART TWO)—CASE II—3750-LB. CONCRETE—$n = 8$



(Instructions continued from preceding page.)

Enter the diagram with the value of k and proceed vertically to an intersection with the sloping index line corresponding to the modified value of p_o. From this intersection pass horizontally to the left or right marginal scale and read off the value of L. Solve formulas (31) and (32) for the stresses in the concrete and reinforcing steel. (See also general note under Diagram 20.)

DIAGRAM 66.—MAXIMUM SIZE OF STIRRUP BARS



This diagram is based on the requirement that a tensile stress of 16,000 lb. per sq. in. in the stirrup or web reinforcement bar can be developed by bond on the surface area of the bar embedded within the upper or lower half of the beam at a unit stress of 0.04 $f'c$ for plain bars or of 0.05 fc' for deformed bars. The diagram is based on the usual U-shaped stirrups, anchored at one face of the beam by bending around the longitudinal bars and at the other by means of hooked ends. The diagram is based on a length of hook such that if straightened out, the end of the bar would project 5 in. beyond the surface of the beam. In case stirrup bars larger than the values obtained by this diagram are used the length of hook must be increased or the tensile stress reduced to that which can be developed by bond on the surface of the bars, including the hook, embedded within the half depth of beam.

For vertical stirrups the maximum size of stirrup for any effective depth of beam is obtained directly from that rectangle in the diagram corresponding to the strength of concrete to be used. For inclined web members the maximum size for any effective depth will be larger than that given by the diagram and may be computed by the formula—

$$Di = Dv\,\frac{(d-1)\ \text{cosec}\ \alpha + 6}{d + 5}$$

in which Di = maximum size of inclined web member.
Dv = maximum size of vertical stirrup by diagram 66.
d = effective depth of beam.
α = angle between inclined web member and the horizontal.

TABLE 67.—VALUES OF NA_v FOR U-SHAPED STIRRUPS

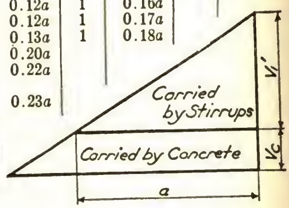
Number of Stirrups at One End	Size of Stirrup					
	¼ in. round	⅜ in. round	½ in. round	½ in. square	⅝ in. round	¾ in. round
20	1.96	4.42	7.85	10.00	12.27	17.67
19	1.87	4.20	7.46	9.50	11.66	16.79
18	1.77	3.97	7.07	9.00	11.04	15.90
17	1.67	3.75	6.67	8.50	10.43	15.02
16	1.57	3.53	6.28	8.00	9.82	14.14
15	1.47	3.31	5.89	7.50	9.20	13.25
14	1.37	3.09	5.50	7.00	8.59	12.37
13	1.28	2.87	5.10	6.50	7.98	11.49
12	1.18	2.65	4.71	6.00	7.36	10.60
11	1.08	2.43	4.32	5.50	6.75	9.72
10	0.98	2.21	3.93	5.00	6.14	8.84
9	0.88	1.99	3.53	4.50	5.52	7.95
8	0.79	1.77	3.14	4.00	4.91	7.07
7	0.69	1.55	2.75	3.50	4.30	6.19
6	0.59	1.32	2.36	3.00	3.68	5.30
5	0.49	1.10	1.96	2.50	3.07	4.42
4	0.39	0.88	1.57	2.00	2.45	3.53
3	0.29	0.66	1.18	1.50	1.84	2.65
2	0.20	0.44	0.79	1.00	1.23	1.77
1	0.10	0.22	0.39	0.50	0.61	0.88

INSTRUCTIONS FOR USE.—The areas given in this table are the right sectional areas of two legs each for the size of stirrup bar shown in the heading and for the number of stirrups given at the left.

The number of stirrups is the number at *each* end for a uniformly loaded beam, or the number of stirrups within the base distance a, in Fig. 7 or 9, of the trapezoid under the shear curve for which stirrups are being designed by formulas (117) or (118).

TABLE 68.—SPACING OF U-SHAPED STIRRUPS—CASES II AND III

Number of Stirrups at One End	Distance, First Stirrup to Face of Support	Spacing, Center to Center of Stirrups, in Terms of a									
		1st Group		2nd Group		3rd Group		4th Group		5th Group	
		No.	Spacing	No.	Spacing	No.	Spacing	No.	Spacing	No.	Spacing
20............	0.013a	8	0.03a	7	0.04a	2	0.06a	1	0.08a	1	0.11a
19............	0.013a	7	0.03a	6	0.04a	3	0.06a	1	0.08a	1	0.12a
18............	0.014a	6	0.03a	5	0.04a	4	0.06a	1	0.08a	1	0.12a
17............	0.015a	5	0.03a	5	0.04a	4	0.06a	1	0.09a	1	0.13a
16............	0.016a	3	0.03a	5	0.04a	5	0.06a	1	0.09a	1	0.13a
15............	0.017a	2	0.03a	5	0.04a	4	0.06a	2	0.08a	1	0 14a
14............	0.018a	5	0.04a	4	0.05a	2	0.08a	1	0.09a	1	0 14a
13............	0.019a	4	0.04a	3	0.05a	3	0.08a	1	0.09a	1	0.14a
12............	0.021a	6	0.05a	3	0.07a	1	0.12a	1	0.15a		
11............	0.023a	5	0.05a	3	0.08a	1	0.12a	1	0.15a		
10............	0.025a	3	0.05a	4	0.08a	1	0.12a	1	0.16a		
9............	0.028a	3	0.06a	3	0.09a	1	0.12a	1	0.17a		
8............	0.032a	2	0.07a	3	0.09a	1	0.13a	1	0.18a		
7............	0.036a	3	0.08a	2	0.13a	1	0.20a				
6............	0.04a	3	0.10a	1	0.15a	1	0.22a				
5............	0.05a	2	0.12a	1	0.16a	1	0.23a				
4............	0.07a	2	0.16a	1	0.26a						
3............	0.09a	1	0.21a	1	0.30a						
2............	0.13a	1	0.37a								
1............	0.29a										



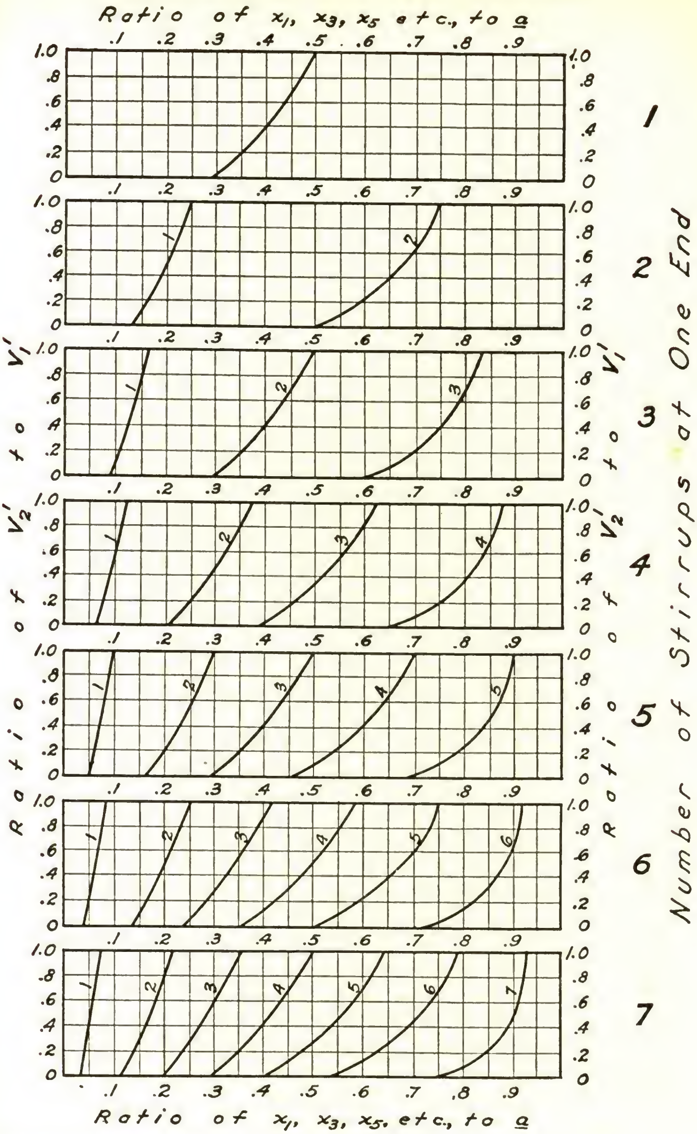
INSTRUCTIONS FOR USE.—Determine from formulas (117) or (118) and Table 67 the value of N, the number of stirrups for one triangle and from formulas (119) or (120) the value of a, the base dimension of the triangle. With these values of N and a enter the table and compute the stirrup spacing.

INSTRUCTIONS FOR USE OF DIAGRAMS 69, 70, 71

These diagrams give stirrup locations for either vertical or inclined stirrups (at the level of the mid-depth of beam), measured in terms of a from the high end of the trapezoid under the shear curve (see Fig. 7 or 9). Enter the proper rectangle, as determined by N, the number of stirrups computed by equation (117) or (118) and Table 67. Take a straight-edged slip of paper and lay it horizontally across this rectangle at the value of $\frac{V'_2}{V'_1}$ determined from the trapezoid under the shear curve for which stirrups are being designed. Mark on the edge of the slip of paper the intersections with the two marginal lines and with each curve numbered from 1 to N. Now place this slip on Diagram 72 horizontally with the value of a as indicated on the margin equal to a from the shear trapezoid. Read off and record the stirrup locations (as measured from high side of the trapezoid) at each of the marks on the slip. The differences between successive stirrup locations give the spacing between stirrups. Additional stirrups must be provided, if necessary, to reduce the maximum spacing to $\frac{3}{4}d$ for beams in which the unit shearing stresses do not exceed $.06f'_c$ or to $\frac{3}{8}d$ for beams with higher shearing stresses. (See 1928 Joint code, Section 803.)

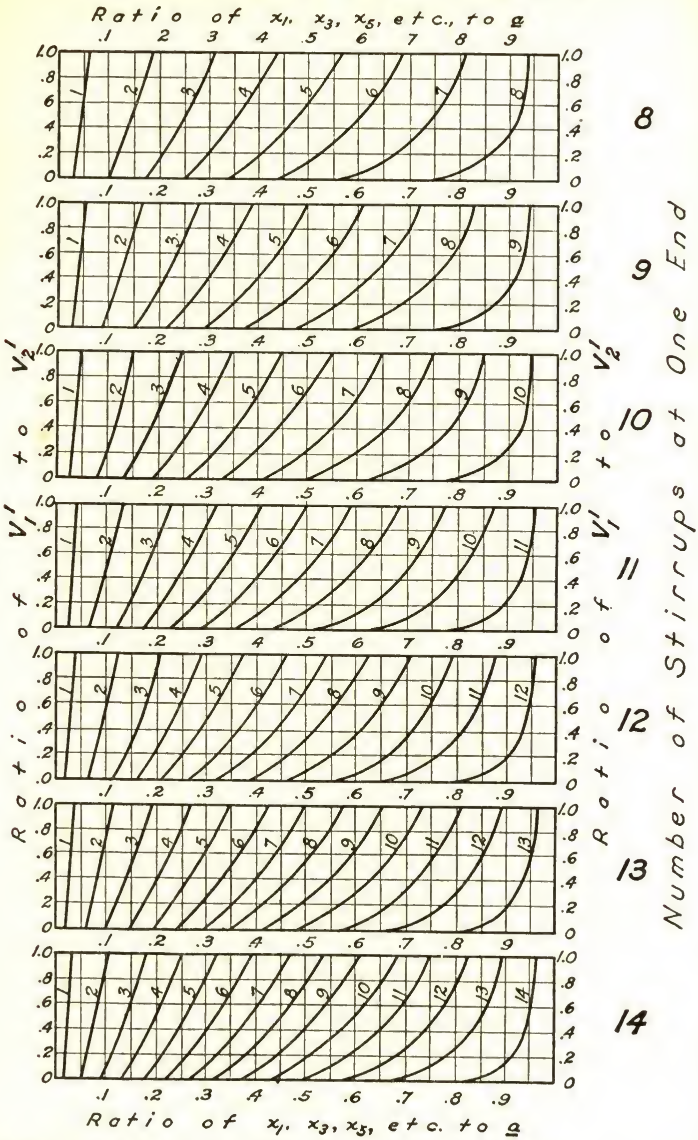
Note.—The process of reading Diagrams 69, 70 and 71 may be simplified by using the loose leaf copy of Diagram 72 (found in the pocket in the rear cover of this edition) in place of a slip of paper as noted above. Fold the loose leaf copy along the horizontal line representing the value of a from the shear trapezoid. Now place the folded edge horizontally across the proper rectangle of Diagrams 69, 70 and 71 for the number of stirrups required and at the proper height for the value of $\frac{v_2'}{v_1'}$. Read off the distances from the high side of the trapezoid to each stirrup location and from these compute the stirrup spacing. Check for additional stirrups required, as noted above, by Section 803 of the Code.

DIAGRAM 69.—SPACING OF 1 TO 7 STIRRUPS



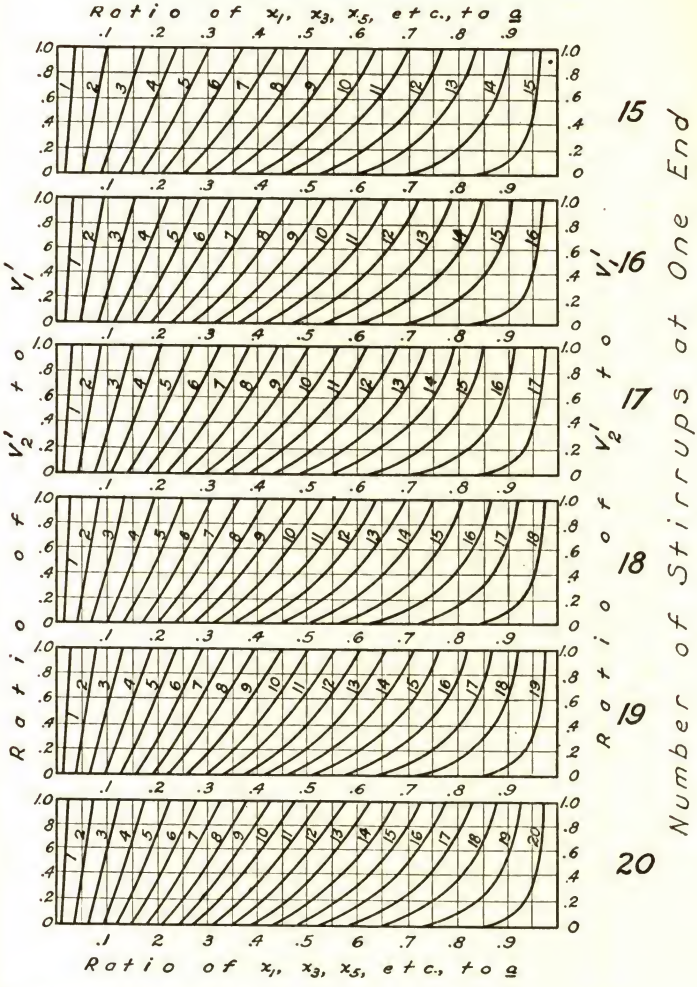
See instructions for use under Table 68.

DIAGRAM 70.—SPACING OF 8 TO 14 STIRRUPS



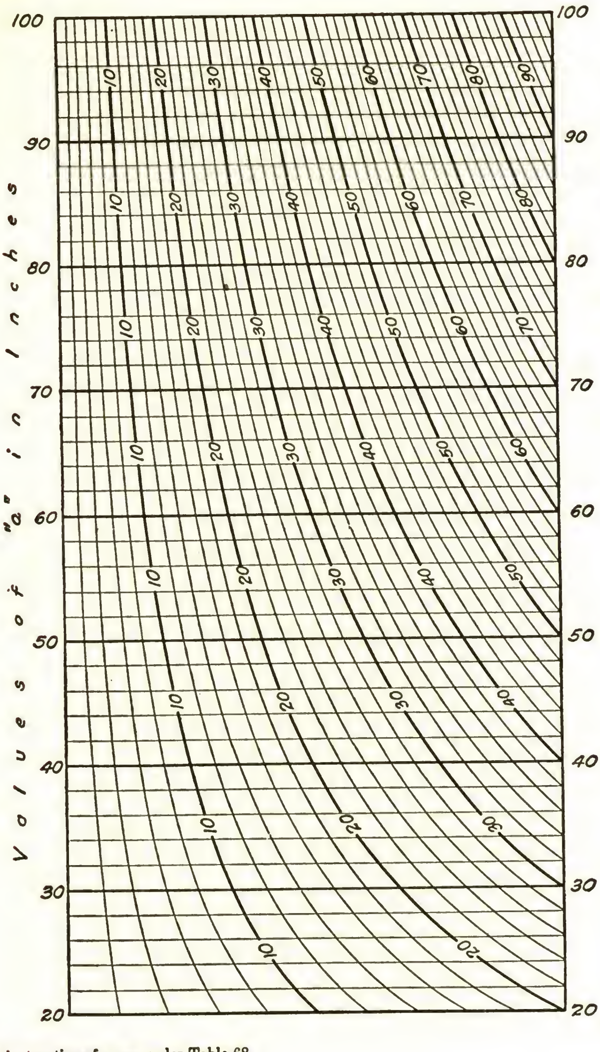
See instructions for use under Table 68.

DIAGRAM 71.—SPACING OF 15 TO 20 STIRRUPS



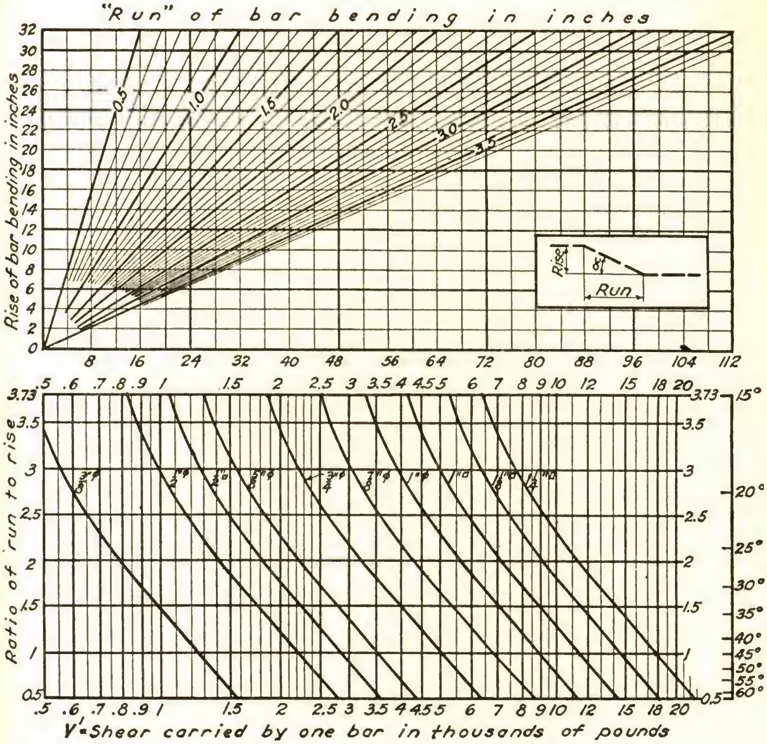
See instructions for use under Table 68.

DIAGRAM 72.—READING CHART FOR DIAGRAMS 69, 70 AND 71



See instructions for use under Table 68.

DIAGRAM 73.—SHEAR VALUES FOR BARS BENT UP IN SINGLE PLANE



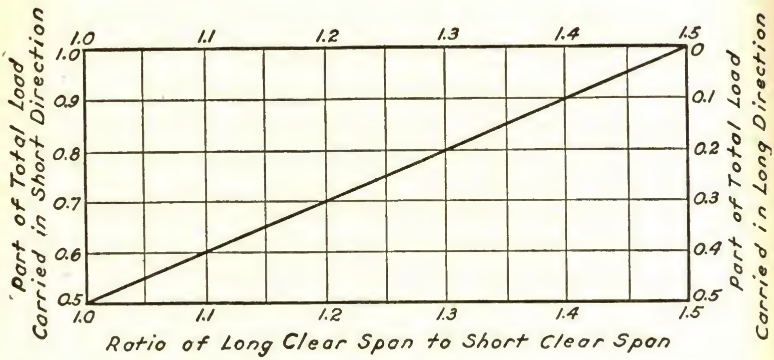
INSTRUCTIONS FOR USE.—This diagram gives designs in accordance with formula (15) in section 803 of the code and excludes any design not in conformity therewith. The upper portion of this diagram is given for convenience in translating "rise" and "run" dimensions into *slope* or ratio of run to rise.

With this ratio enter the lower diagram at the left (or right) margin and proceed horizontally to an intersection with the bar size used. From this intersection drop vertically and read off the shear resistance of one bar on the bottom scale. Multiply this by the number of bars bent up in the single plane to obtain the total shear resistance.

This diagram applies to one or more bars bent up in a single plane at any angle with the horizontal within the limits indicated at the right margin. The indicated shear resistance may be considered effective only in that portion of the beam within which the center ¾ of the bent portion of the bar lies. The total shear resistance of the beam is the sum of resistance of the bent up bars as found above plus the resistance of the concrete by formula (114).

DESIGN OF TWO WAY SLABS SUPPORTED ON BEAMS

DIAGRAM 74.—DISTRIBUTION OF LOAD BETWEEN LONG AND SHORT DIRECTION



INSTRUCTIONS FOR USE.—The same moment coefficients are used for design strips in two-way slabs within the middle half of the slab in each direction as are used for beams under the same conditions of support and restraint. In the outer quarters, the reinforcement parallel to the supporting beam may be reduced to 50 per cent of that in the middle half in the same direction.

For any ratio of length to breadth this diagram gives the proportion of the total dead and live load which the slab must be designed to carry and which it transmits to the supporting beam. The supporting beam must be designed to carry in addition to its own weight and superimposed live load a uniform load throughout its length equal to the load per foot brought to it by the middle strips on either side.

ARRANGEMENT OF REINFORCEMENT IN FLAT SLAB FLOORS.

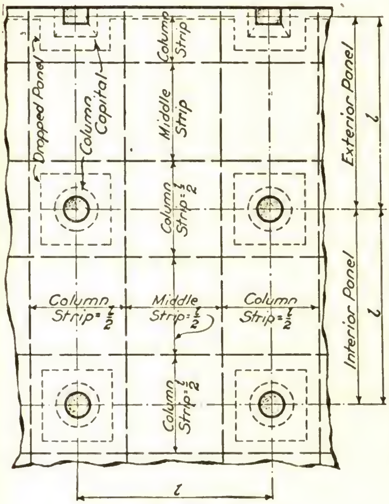


FIG. 14.—TWO-WAY.

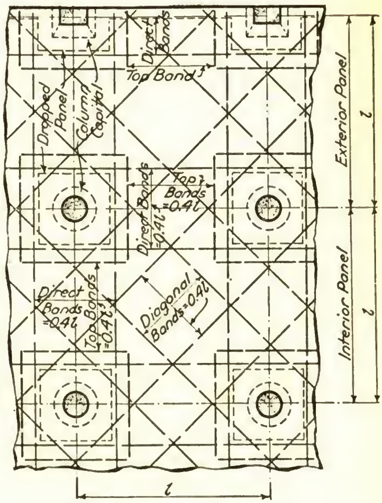


FIG. 15.—FOUR-WAY.

INSTRUCTIONS FOR USE OF DIAGRAMS 77, 79, 81, ETC., TO 91

These diagrams are to be used for square panels, of either two-way or four-way arrangement, surrounded by other interior or exterior panels of the same size, shape and loading. They apply to interior panels when used in conjunction with Table 75 and to exterior panels with full size capitals when used in conjunction with Tables 75 and 76 jointly as described in the notes under Table 76. Diagrams 79, 83, 87 and 91 are based on the use of forms giving column capitals 0.225 l in diameter. Diagrams 77, 81, 85 and 89 are based on the use of metal forms with the usual 6-in. intervals between successive sizes.

In selecting bars to fit the areas determined from these diagrams and Tables 75 and 76 the area actually provided in bent bars must not be reduced as this would result in a deficiency over the column head. It is entirely proper to reduce the actual area in the straight bars of any strip by the amount of the excess in area actually provided in the bent bars of the same strip.

These diagrams are based on a minimum of one inch of fireproofing between all bars and the slab surfaces.

INSTRUCTIONS FOR USE OF DIAGRAMS 78, 80, 82, ETC., TO 92

Enter the diagram at the top with the side dimension of the square panel and drop vertically to an intersection in the upper group of curves with the proper curve for the live load used in design. Read off the volume of concrete by moving horizontally from this intersection to either side scale.

In the same manner drop on the same vertical line to the middle and lower group of curves, successively, and read off the area of formwork and the weight of reinforcing steel on the side scales.

DESIGN COEFFICIENTS FOR FLAT SLAB FLOORS

TABLE 75.—SQUARE INTERIOR PANELS

Diagrams 77, 79, etc., inclusive give slab thickness, column capital diameters, dimensions of square dropped panels and values of A_s for both two-way and four-way flat slabs, with live loads varying from 100 to 300 lb. per sq. ft. and with panel sides from 16 to 24 feet. To complete the design proceed as follows:

The *thickness* of the dropped panel is always equal to one-half of the slab thickness from the diagram.

For *two-way flat slabs* the area of bars in each strip bears the following relation to A_s from the diagrams:

The area of the bent bars in each middle strip is 0.89 A_s.

The area of the straight bars in each middle strip is A_s.

The area of the bent bars in each column strip is 1.56 A_s.

The area of the straight bars in each column strip is 0.78 A_s.

The area of the straight bars, which must be added in the top of the slab over the column head in each column strip is 0.36 A_s.

For *four-way flat slabs* the area of bars in each band, taken at right angles to the direction of the band, bears the following relation to A_s from the diagrams:

The area of bars in each top band, across the direct band, is A_s.

The area of the bent bars in each diagonal band is 0.67 A_s.

The area of the straight bars in each diagonal band is A_s.

The area of the bent bars in each direct band is A_s.

The area of the straight bars in each direct band is 1.22 A_s.

The shearing and compressive unit stresses and the amount of reinforcement at the column head will not require computation where these diagrams are used in the manner set forth above.

The coefficients apply to interior panels surrounded by other interior or exterior panels approximately the same size and shape and subjected to the same loading.

DESIGN COEFFICIENTS FOR FLAT SLAB FLOORS

TABLE 76.—SPECIAL REQUIREMENTS FOR SQUARE EXTERIOR PANELS

For *two-way flat slabs* the area of bars in each strip bears the following relation to A_s from the diagrams:

The area of bent bars for each middle strip perpendicular to the wall is 1.11 A_s.

The area of straight bars for the same strip is 1.12 A_s.

The area of bent bars for each column strip perpendicular to the wall is 1.95 A_s.

The area of the straight bars for the same strip is 0.98 A_s.

The area of the straight bars, which must be added in the top of the slab over the exterior column head is 1.18 A_s.

No additional straight bars in the column strip perpendicular to the wall will need to be added over the interior column head adjacent to the exterior panel.

For *four-way flat slabs* the area of bars in each band bears the following relation to A_s from the diagrams:

The area of bars in the top band across the direct band at the wall side of the panel will be 0.625 A_s.

The area of the bent bars in each diagonal band will be 1.13 A_s.

The area of the straight bars in each diagonal band will be 0.75 A_s.

The area of the straight bars which must be added in the top over the exterior column head in each diagonal band will be 0.22 A_s.

The area of the bent bars in each direct band perpendicular to the wall is 1.67 A_s.

The area of the straight bars in each direct band perpendicular to the wall is 1.11 A_s.

NOTE.—The slab and dropped panel thicknesses are taken from Diagrams 77, 79, etc., and are the same as for interior panels of the same size, shape and loading.

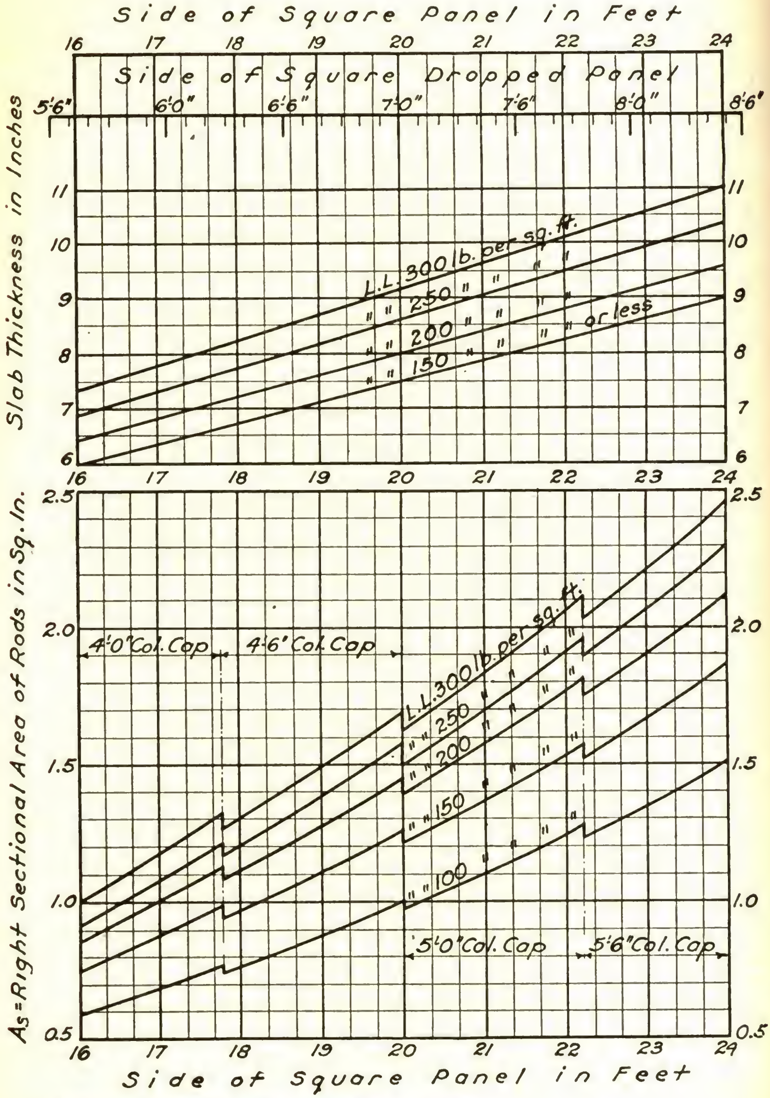
These coefficients apply only to exterior panels surrounded on three sides by other exterior or interior panels of the same size and shape and subject to the same loading. The column capitals at the wall must be the same size as for interior panels, except as cut off by the building line, in order for this table and the accompanying diagrams 77, 79, etc., to apply.

The steel area in strips parallel to the wall will be identical to the steel area found by Table 75 for the corresponding strips of an interior panel, except that the strip lying along the wall will have only a portion of the width of an interior strip and will be designed in accordance with section 1011 of the 1928 Joint code.

Steel areas in the strips in *both* directions in corner panels will be determined solely by this table.

DESIGN OF TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS

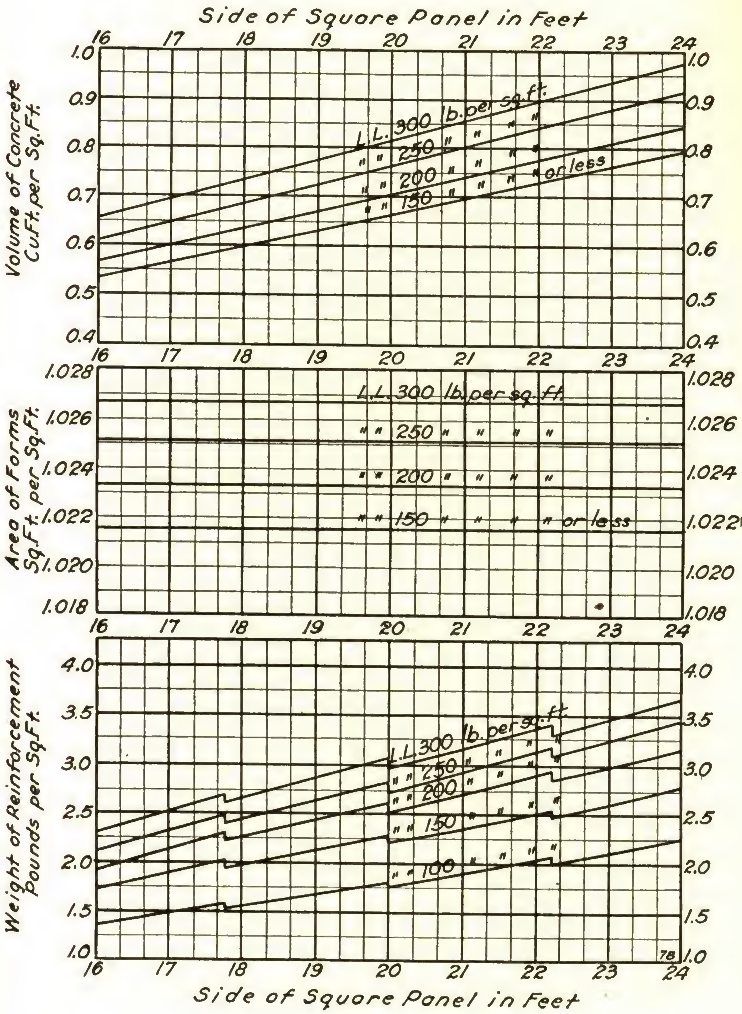
DIAGRAM 77.—2000-LB. CONCRETE—METAL COLUMN FORMS



For instructions for use see general note under Fig. 14, page 115.

QUANTITIES FOR TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS

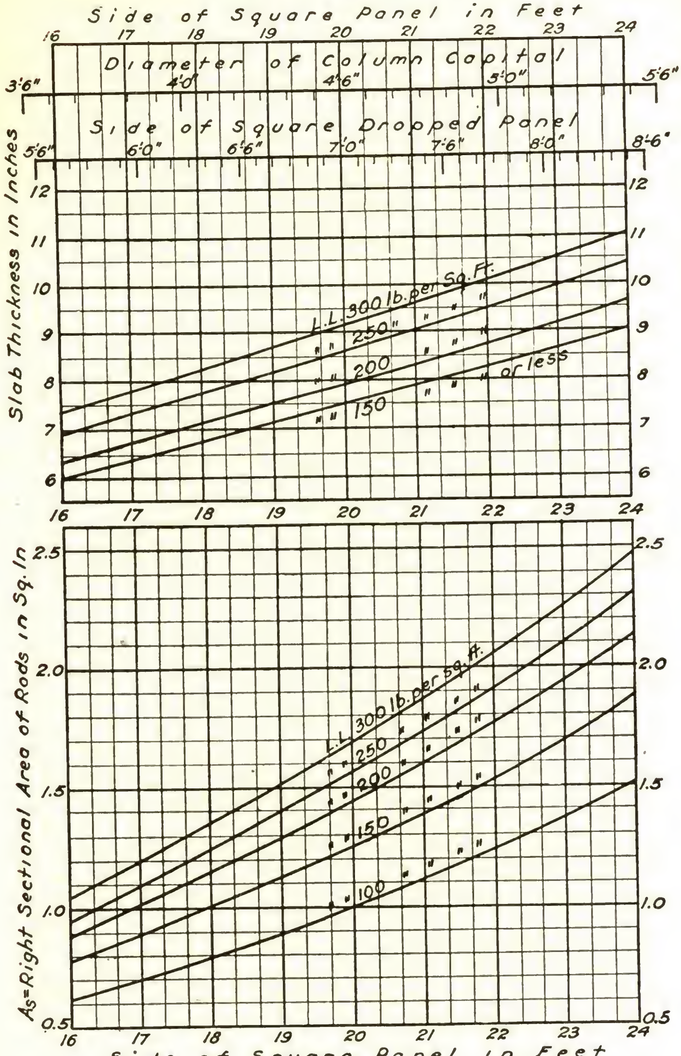
DIAGRAM 78.—2000-LB. CONCRETE—METAL COLUMN FORMS



For instructions for use see general note under Fig. 14, page 115.

DESIGN OF TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS

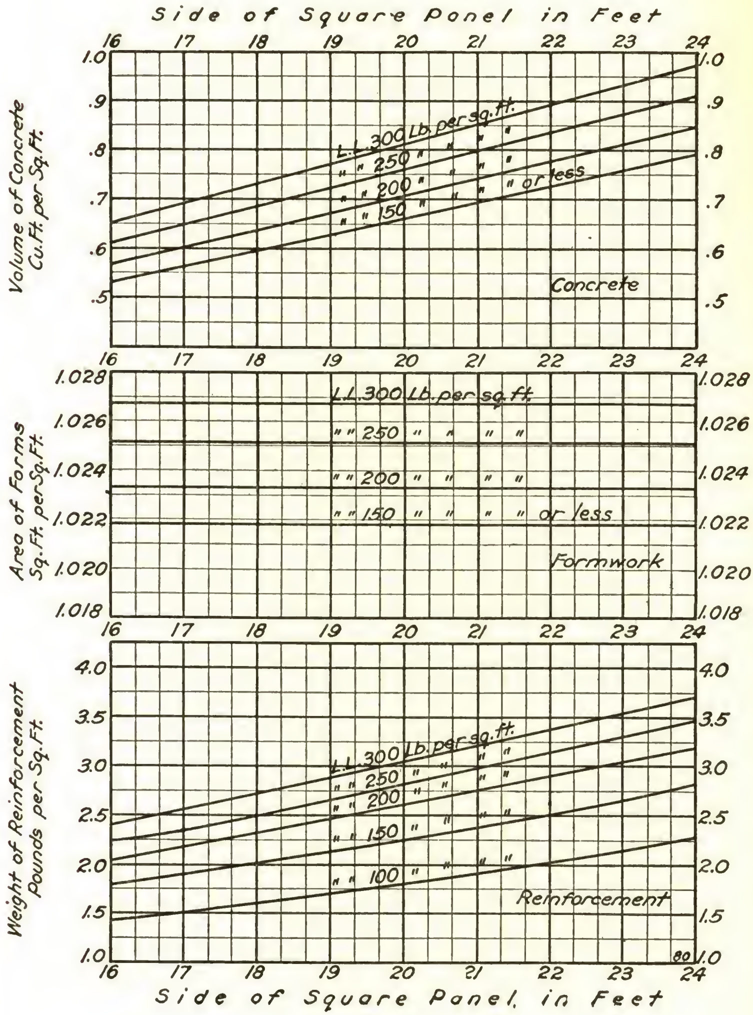
DIAGRAM 79.—2000-LB. CONCRETE—0.225l CAPITAL



For instructions for use see general note under Fig. 14, page 115.

QUANTITIES FOR TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS

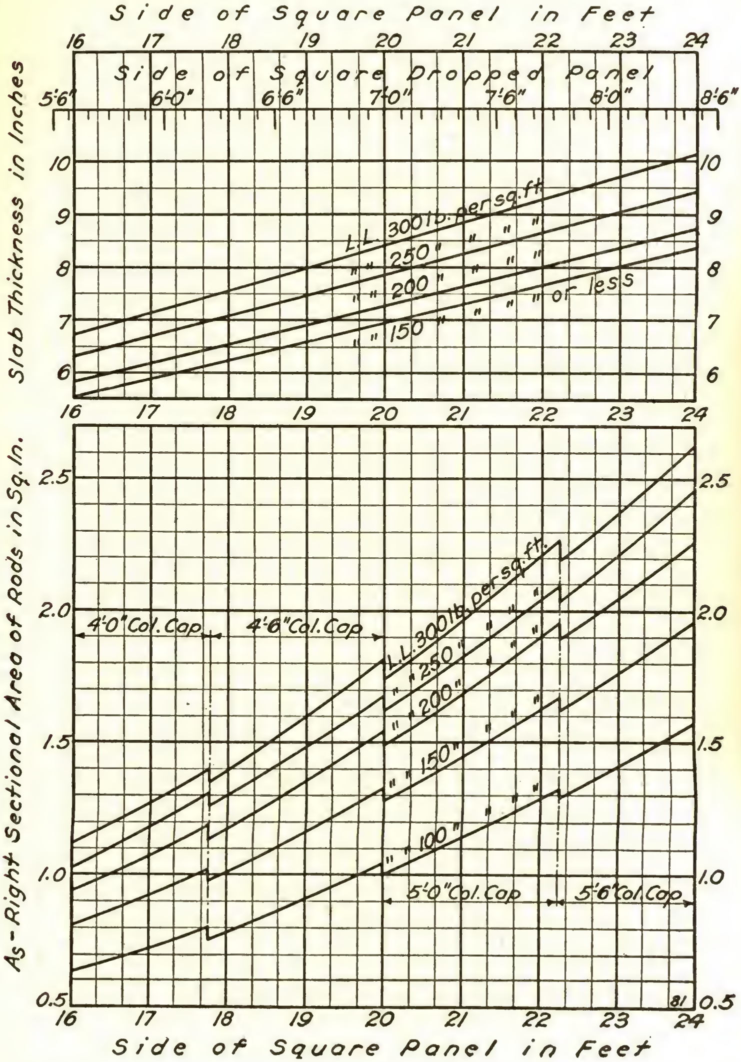
DIAGRAM 80.—2000-LB. CONCRETE—0.225l CAPITAL



For instructions for use see general note under Fig. 14, page 115.

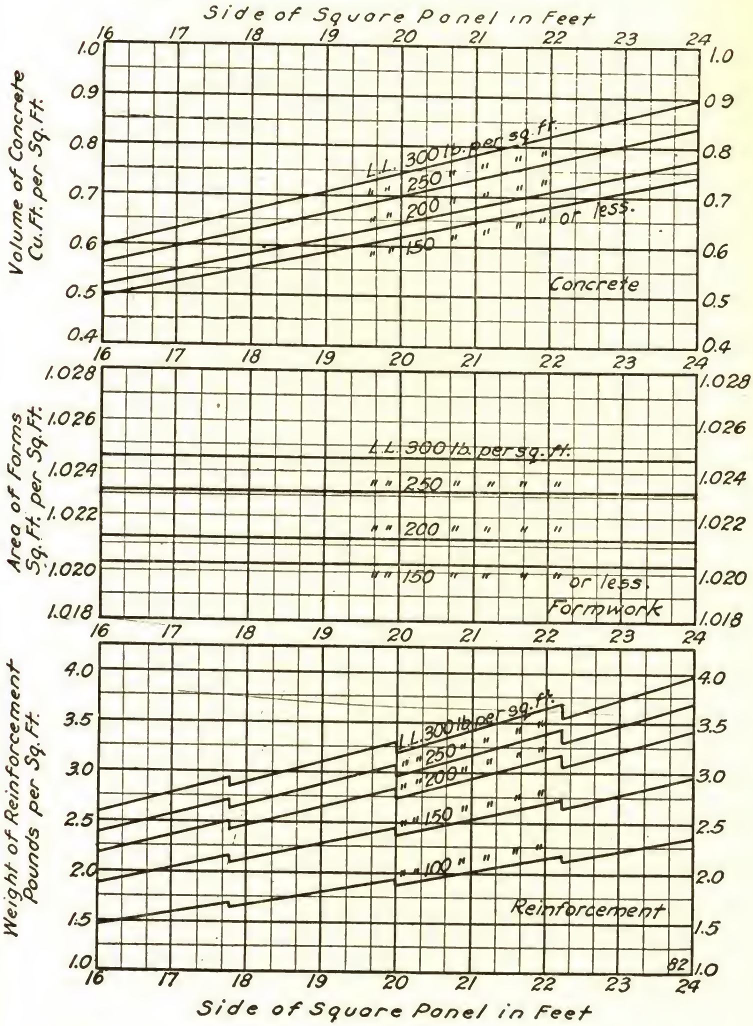
DESIGN OF TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS

DIAGRAM 81.—2500-LB. CONCRETE—METAL COLUMN FORMS



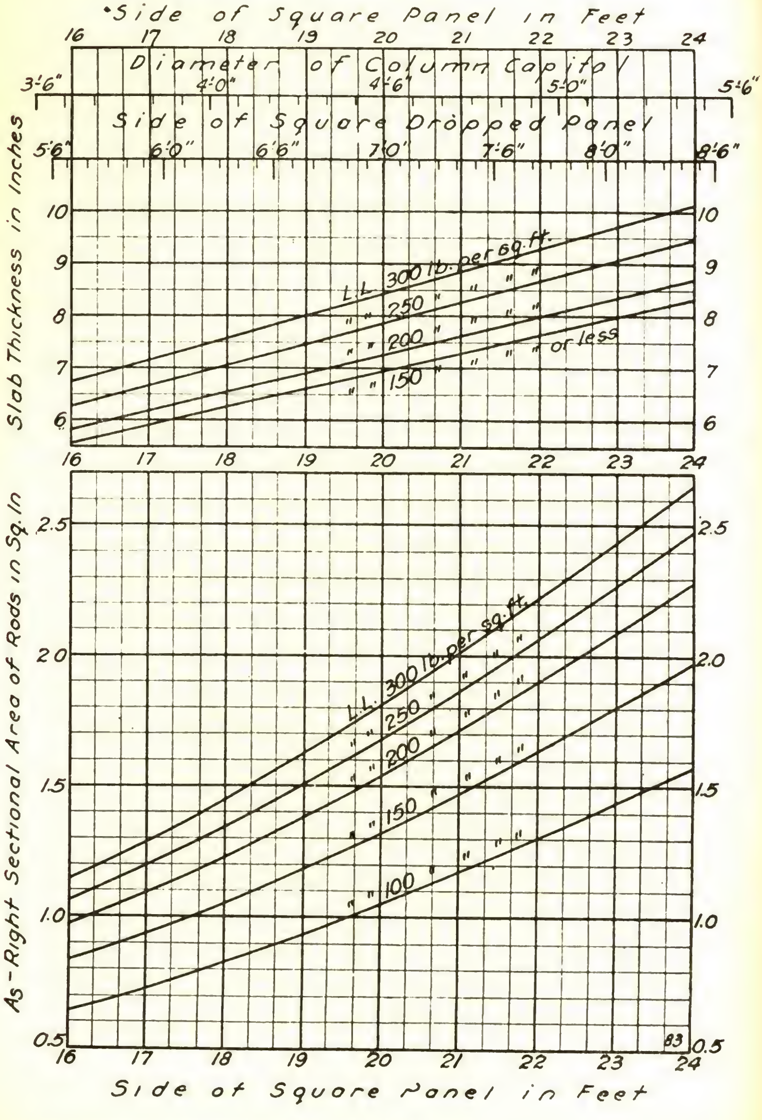
For instructions for use see general note under Fig. 14, page 115.

QUANTITIES FOR TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS
DIAGRAM 82.—2500-LB. CONCRETE—METAL COLUMN FORMS



For instructions for use see general note under Fig. 14, page 115.

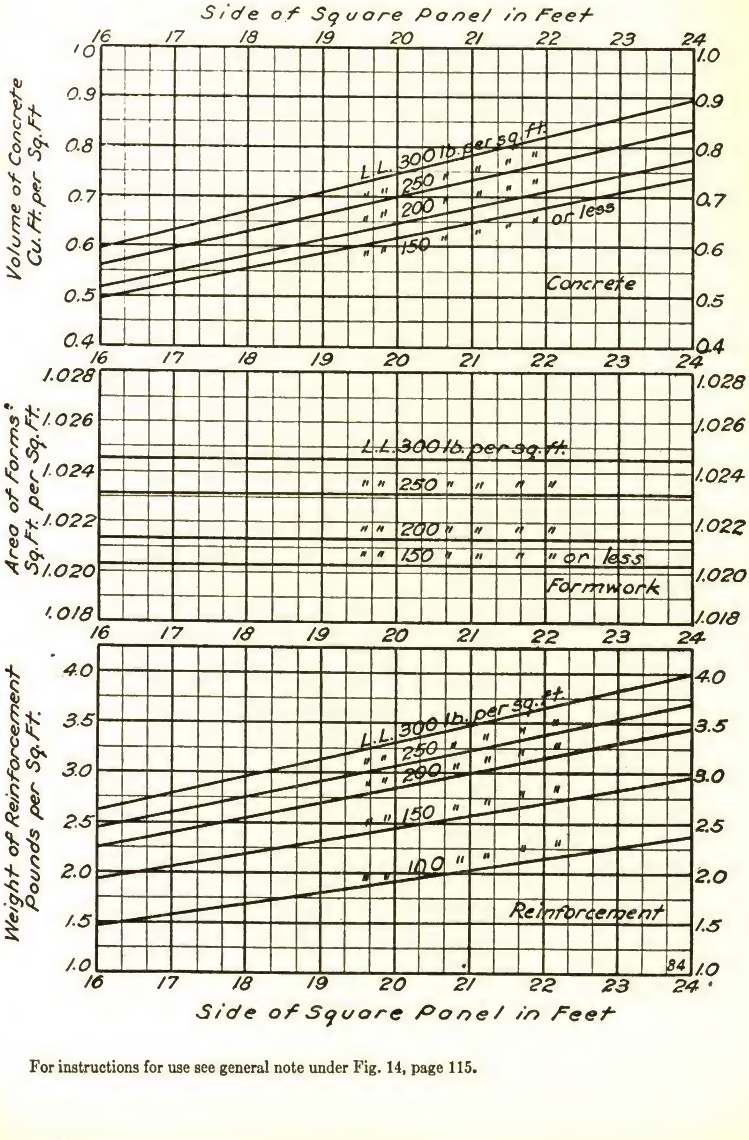
DESIGN OF TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS
DIAGRAM 83.—2500 LB. CONCRETE—0.225*l* CAPITAL



For instructions for use see general note under Fig. 14, page 115.

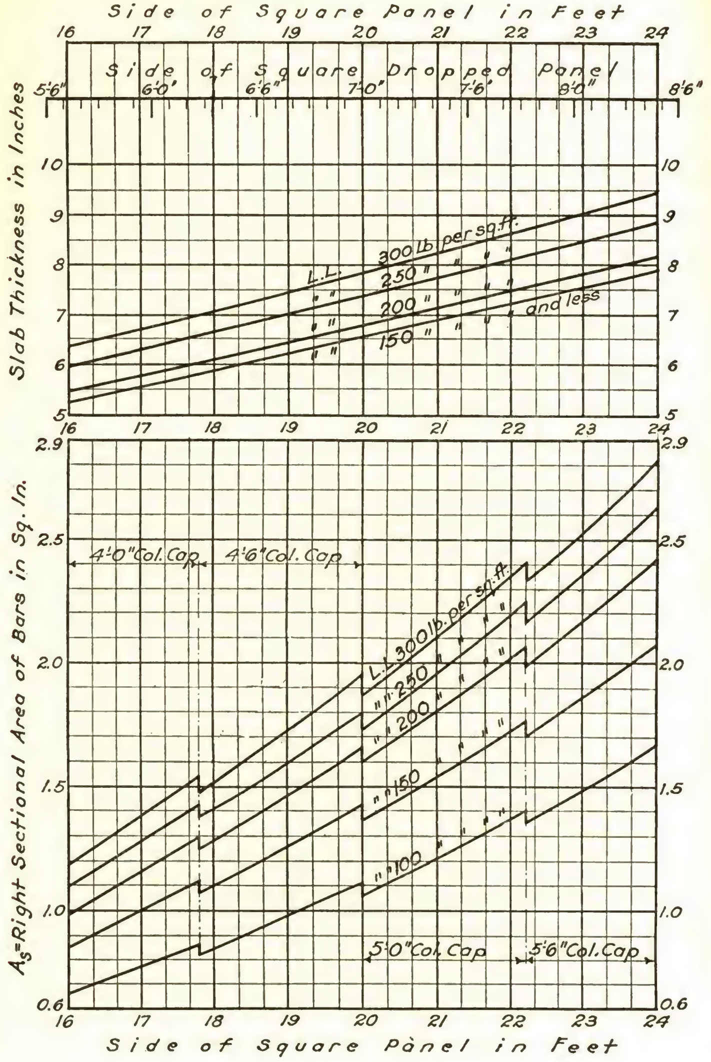
QUANTITIES FOR TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS

DIAGRAM 84.—2500-LB. CONCRETE—0.225l CAPITAL



For instructions for use see general note under Fig. 14, page 115.

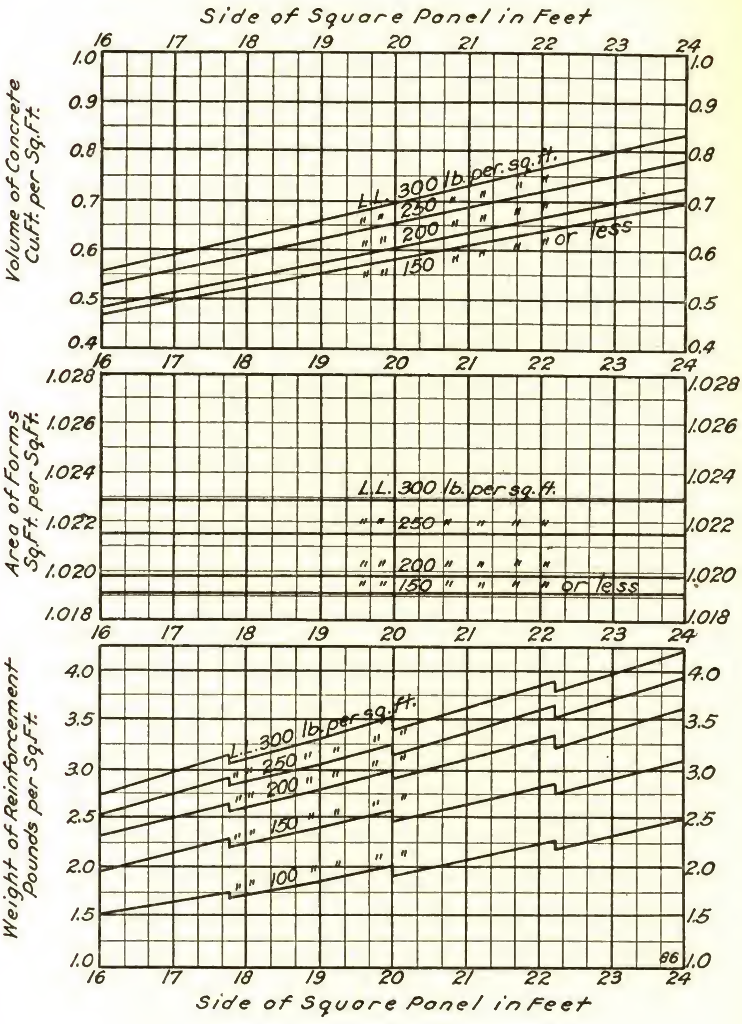
DESIGN OF TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS
DIAGRAM 85.—3000-LB. CONCRETE—METAL COLUMN FORMS



For instructions for use see general note under Fig. 14, page 115

QUANTITIES FOR TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS

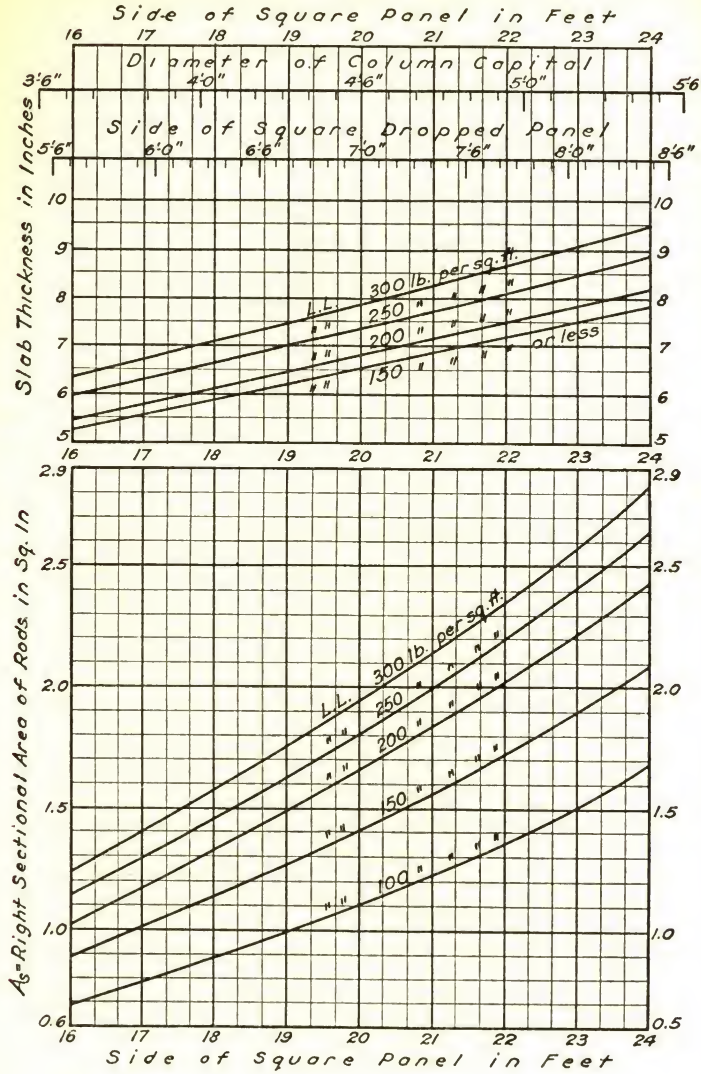
DIAGRAM 86.—3000-LB. CONCRETE—METAL COLUMN FORMS



For instructions for use see general note under Fig. 14, page 115.

DESIGN OF TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS

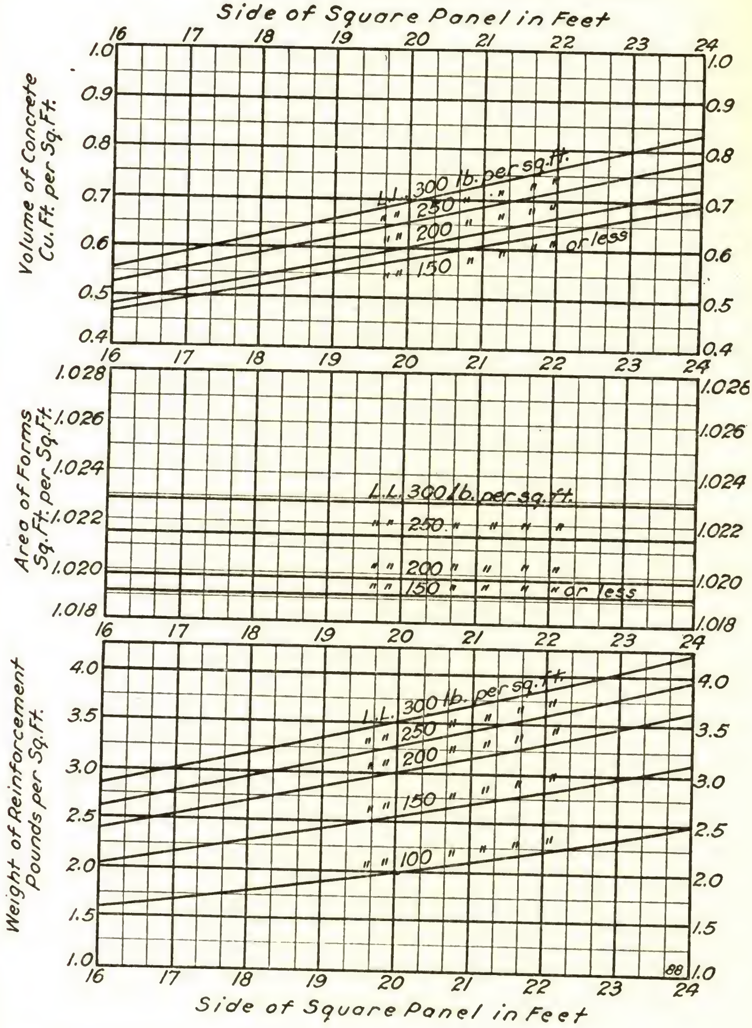
DIAGRAM 87.—3000-LB. CONCRETE—0.225l CAPITAL



For instructions for use see general note under Fig. 14, page 115.

QUANTITIES FOR TWO-WAY AND FOUR-WAY FLAT SLAB FLOORS

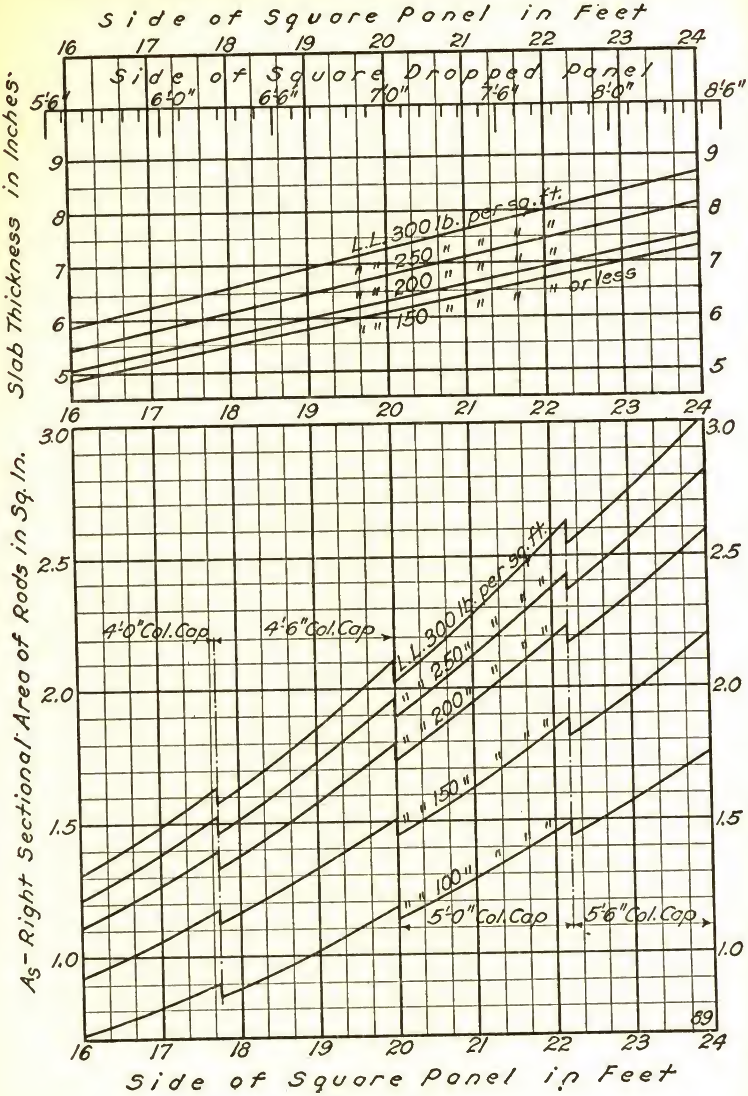
DIAGRAM 88.—3000-LB. CONCRETE—0.225l CAPITAL



For instructions for use see general note under Fig. 14, page 115.

DESIGN OF TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS

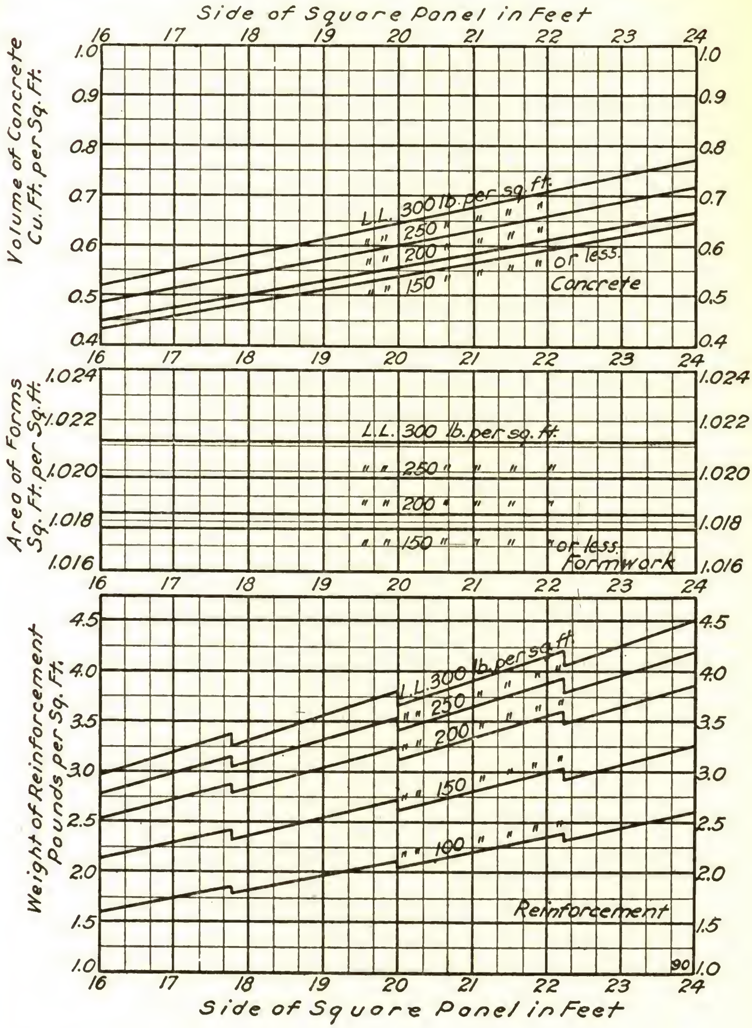
DIAGRAM 89.—3750-LB. CONCRETE—METAL COLUMN FORMS



For instructions for use see general note under Fig. 14, page 115.

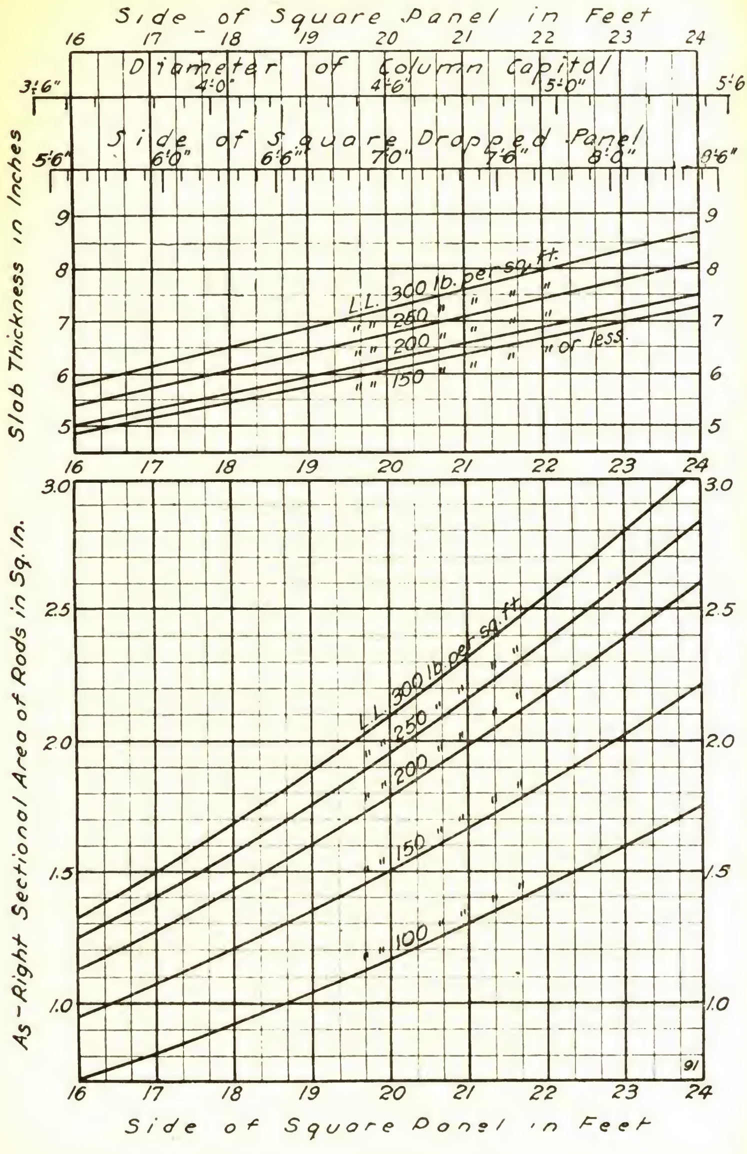
QUANTITIES FOR TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS

DIAGRAM 90.—3750-LB. CONCRETE—METAL COLUMN FORMS



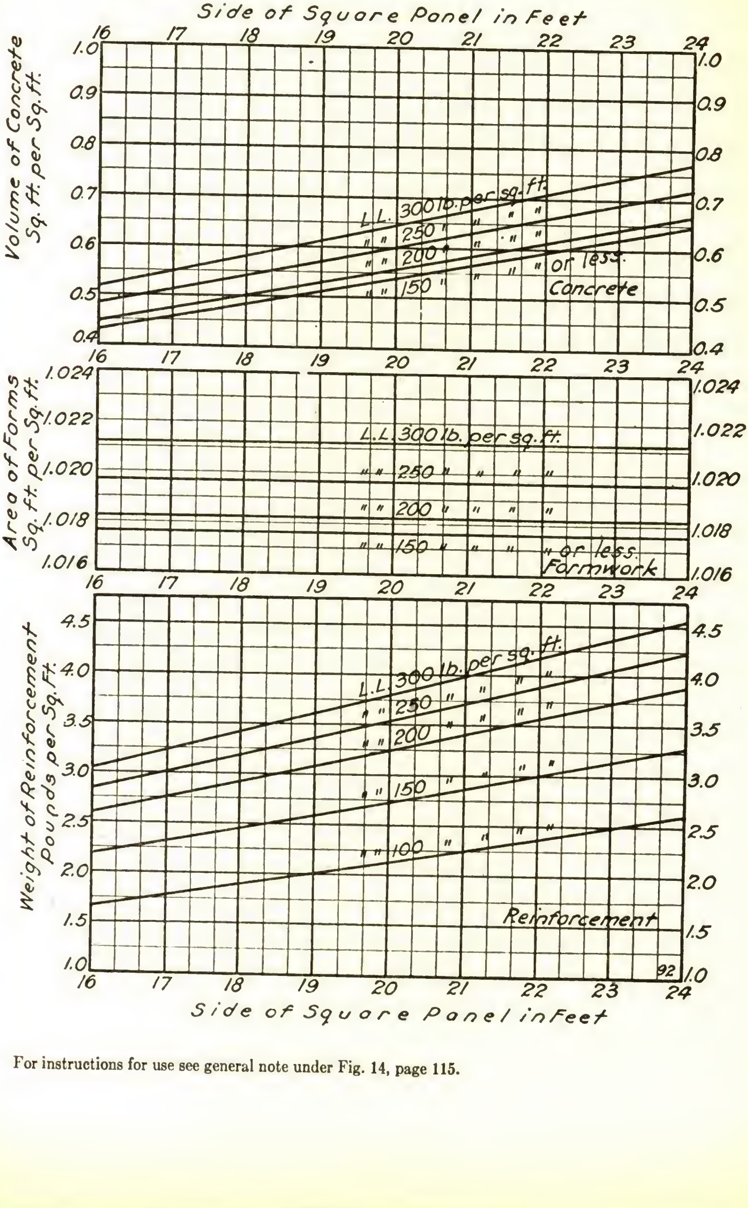
For instructions for use see general note under Fig. 14, page 115.

DESIGN OF TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS
DIAGRAM 91.—3750 LB. CONCRETE—0.225l CAPITAL



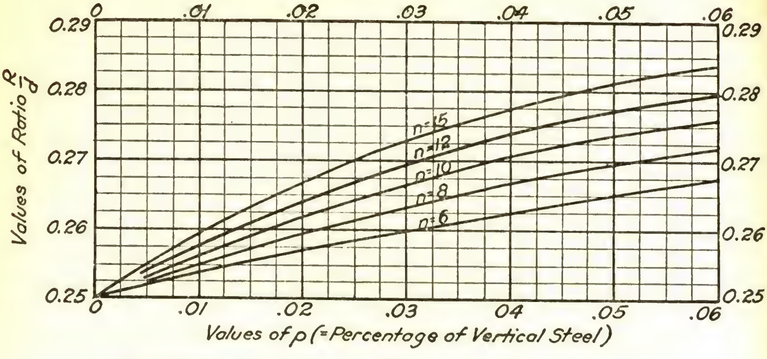
For instructions for use see general note under Fig. 14, page 115.

QUANTITIES FOR TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS
DIAGRAM 92.—3750-LB. CONCRETE—0.225l CAPITAL

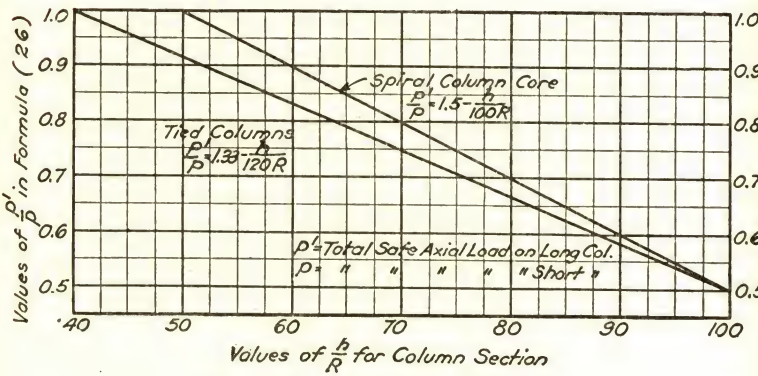


For instructions for use see general note under Fig. 14, page 115.

DIAGRAM 93.—LOAD REDUCTION IN LONG COLUMNS



RADIUS OF GYRATION IN TERMS OF CORE DIAMETER



LOAD FACTOR FOR LONG COLUMNS

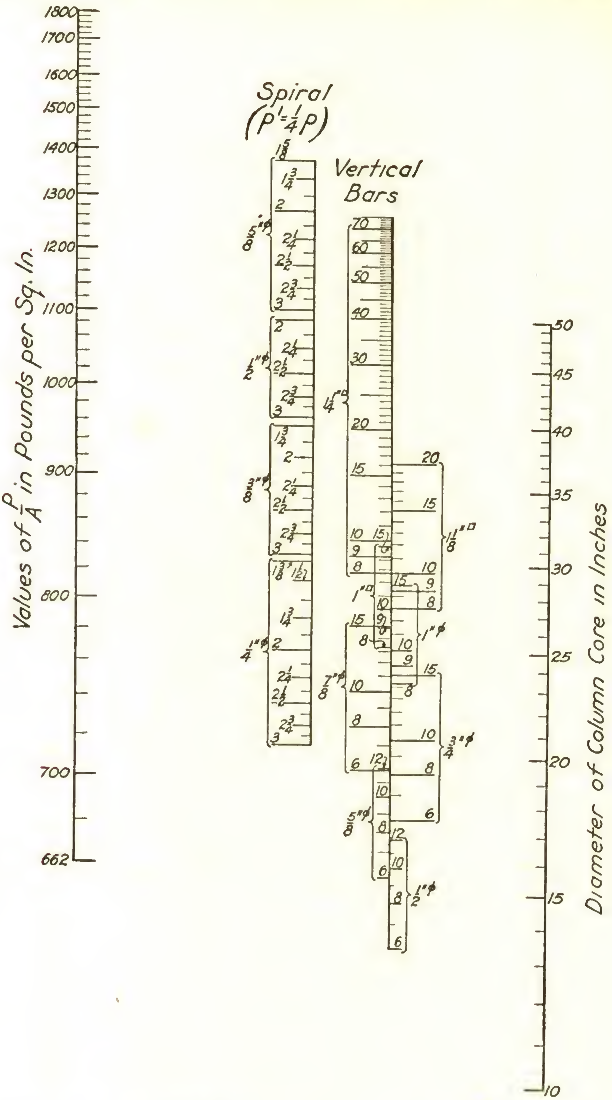
INSTRUCTIONS FOR USE.—This diagram is based on a reduction in load for columns in which the unsupported length exceeds eleven times the least dimension of the column section.

The upper portion gives values of R, the radius of gyration of the transformed section, in terms of the core diameter of a spiral column as used in design. For a tied column the value of R may be taken without considerable waste as that of the concrete alone or as 0.29 times the least dimension of the rectangular column.

The lower portion of the diagram gives the ratio of the load permitted on a long column to that permitted on an ordinary column by the formulas appearing in the diagram. Enter the diagram with the value of h, the unsupported column length divided by R the radius of gyration, move vertically to the proper sloping index line and then horizontally to the marginal scale and read off the allowable proportion of load.

Composite columns are governed by the same load-reduction formula as spiral columns.

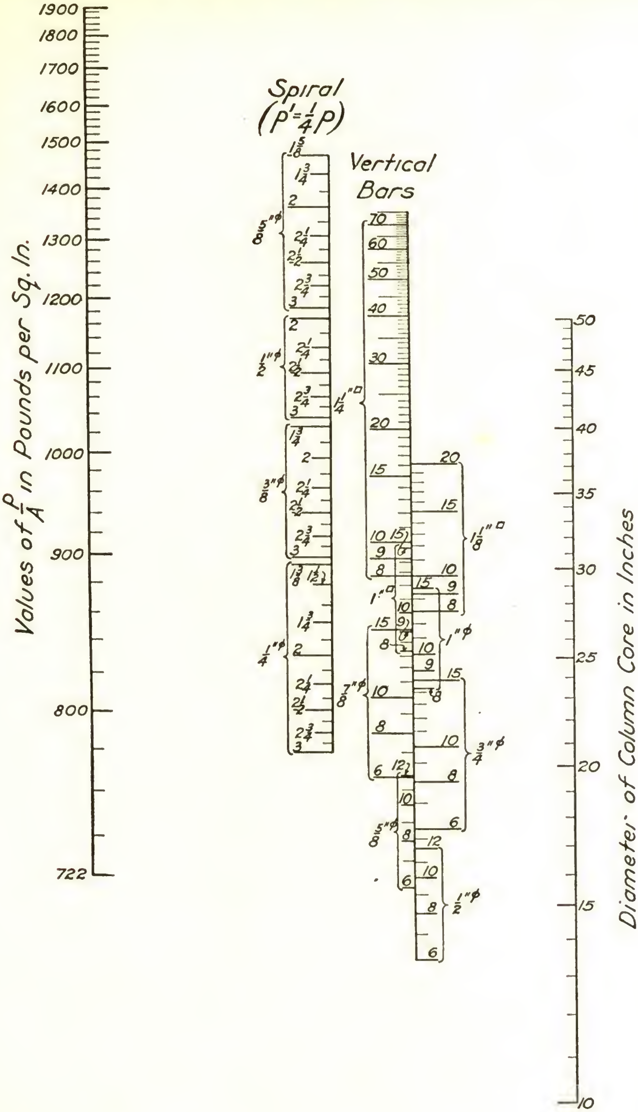
DIAGRAM 94.—DESIGN OF SPIRAL COLUMN—2000-LB. CONCRETE



See instructions for use under Diagram 100.

$p = .04$ when $\frac{P}{A} = 1280$. (See Section 1103-b of the code.)

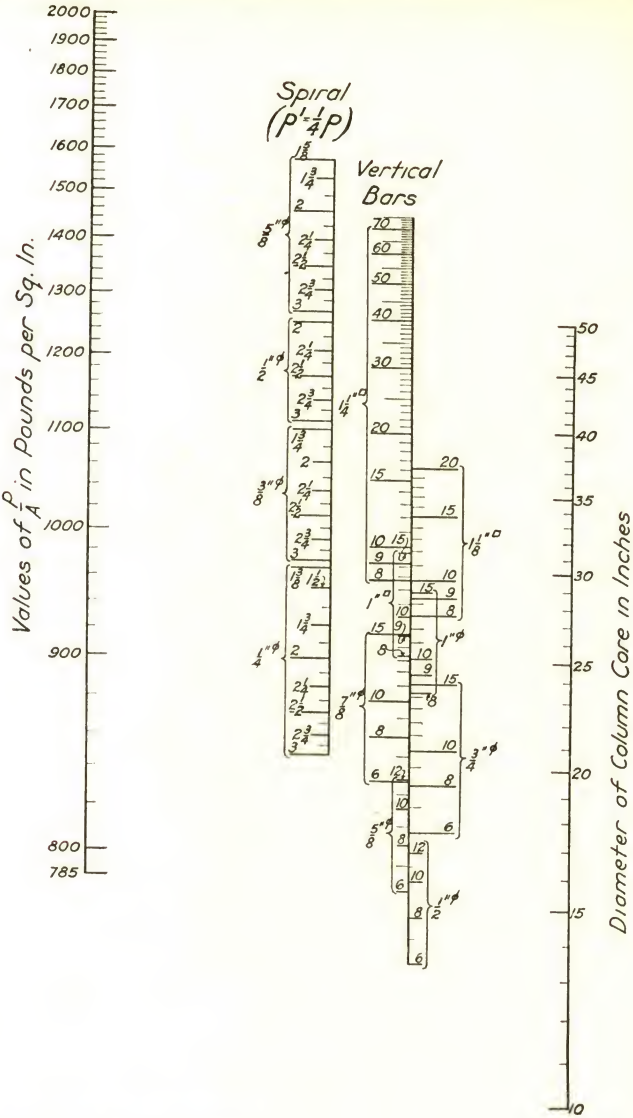
DIAGRAM 95.—DESIGN OF SPIRAL COLUMNS—2500-LB. CONCRETE



See instructions for use under Diagram 100.

$p = .04$ when $\dfrac{P}{A} = 1370$. (See Section 1103-b of the code.)

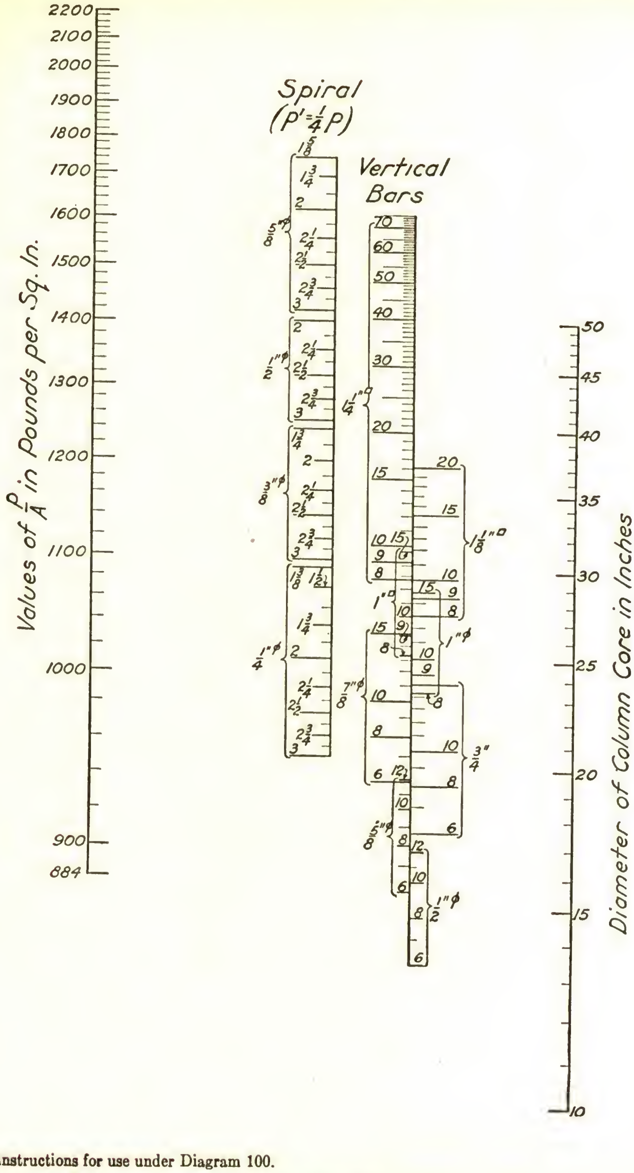
DIAGRAM 96.—DESIGN OF SPIRAL COLUMNS—3000-LB. CONCRETE



See instructions for use under Diagram 100

$p = .04$ when $\frac{P}{A} = 1470$. (See Section 1103-b of the code.)

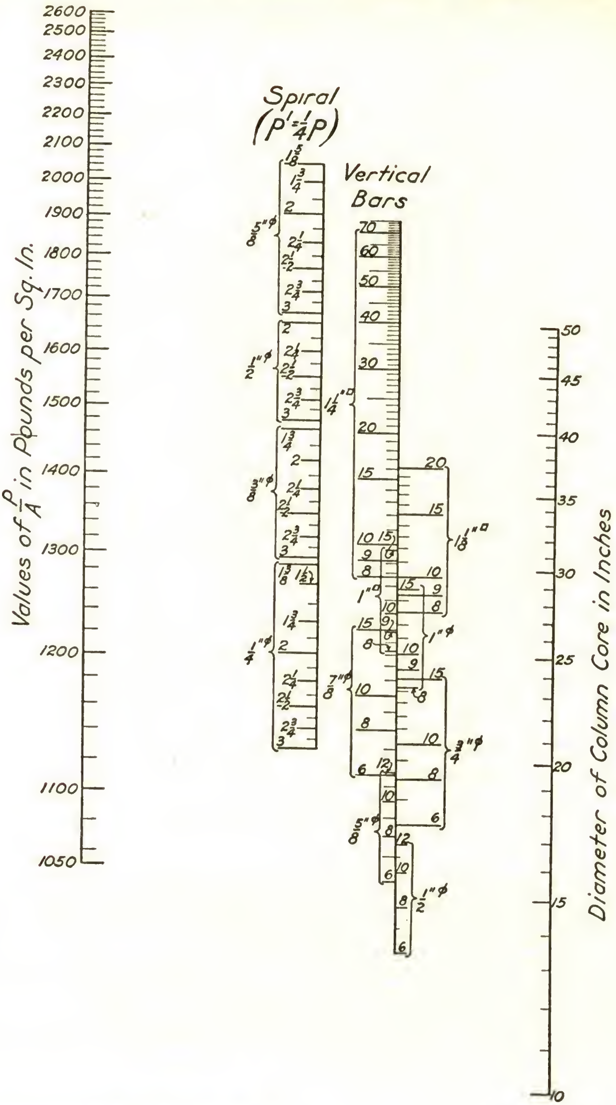
DIAGRAM 97.—DESIGN OF SPIRAL COLUMNS—3750-LB. CONCRETE



See instructions for use under Diagram 100.

$p = .04$ when $\frac{P}{A} = 1630$. (See Section 1103-b of the code.)

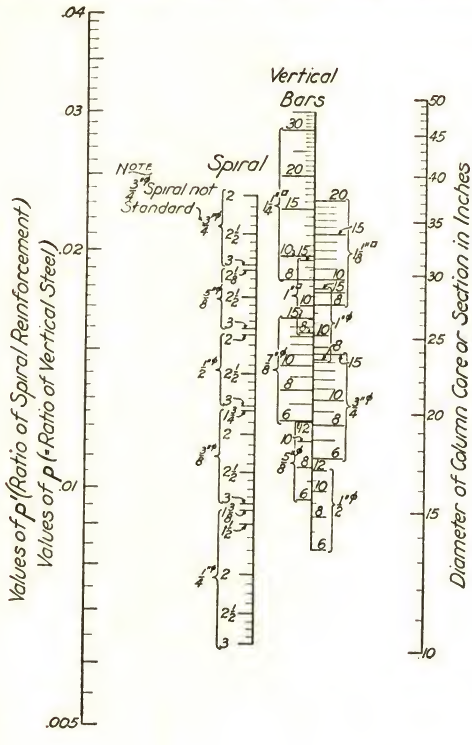
DIAGRAM 98.—DESIGN OF SPIRAL COLUMNS—5000-LB. CONCRETE



See instructions for use under Diagram 100.

$p = .04$ when $\frac{P}{A} = 1920$. (See Section 1103-b o the code.)

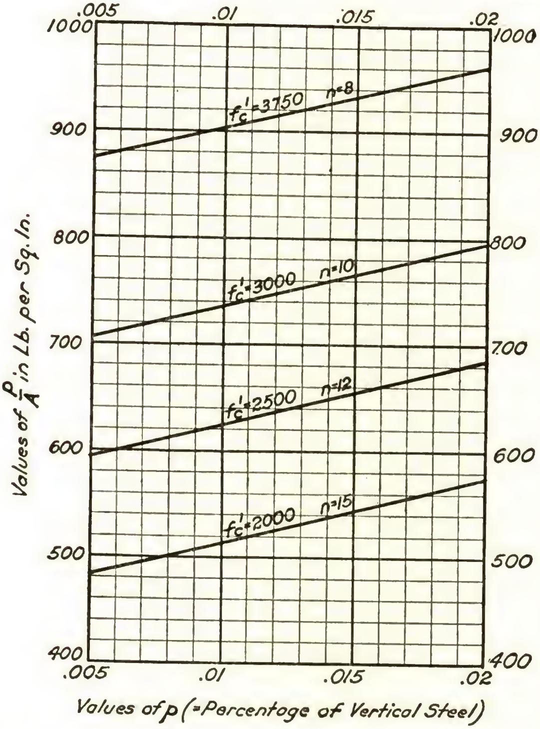
DIAGRAM 99.—SELECTION CHART FOR COLUMN REINFORCEMENT



INSTRUCTIONS FOR USE.—This diagram is really two diagrams superimposed for convenience—a spiral selection diagram made up of the left, left-center and right scales and a vertical bar selection diagram made up to the left, right-center and right scales. It is to be used in connection with Diagram 100 to select vertical bars in *round* tied columns and in the design of composite columns

This is an alignment chart, read by the aid of a straight edge, preferably the edge of a celluloid triangle on account of its transparency. Set the straight edge on the right hand scale to the diameter of the effective column section and on the left scale to the required percentage of *spiral* reinforcement and read off the spiral wire and pitch at the intersection of the straight edge with the *left-cen'er* scale. For the vertical bars, set the straight edge on the right hand scale as before and on the left hand scale to the percentage of *vertical* reinforcement. Read off the number and size of vertical bars at the intersection of the straight edge with the *right-center* scale.

DIAGRAM 100.—DESIGN OF TIED COLUMNS
2000-LB., 2500-LB., 3000-LB. AND 3750-LB. CONCRETE



INSTRUCTIONS FOR USE.—The area A is the full area of the column section including the fireproofing, but the vertical bars are required to be set 2 inches in the clear from the surface.

The minimum vertical steel permitted is 4-⅝ in. rd. bars. The ties must be at least ¼ in. rd. in size, spaced not over 12 inches on centers, and must be so arranged as to afford support to each vertical bar in at least two directions, as called for by Section 1104 of the code.

GENERAL INSTRUCTIONS FOR USE OF DIAGRAMS 94 TO 98

This is an alignment chart, read by the aid of a straight edge, preferably the edge of a celluloid triangle on account of its transparency. Set the edge on the right hand scale to the diameter of the column core and on the left hand scale to the value of P/A (stress in lb. per sq. in. of core area). Read off the *spiral* wire and pitch at the intersection of the straight edge with the *left-cen'er* scale and the number and size of *longitudinal* bars at the intersection with the *right-center* scale. A is found from Table 105.

TABLE 101.—PERCENTAGES AND WEIGHTS OF ¼-IN. ROUND SPIRALS

Core Dia. Inches	PITCH												
	3 in.	2⅞ in.	2¾ in.	2⅝ in.	2½ in.	2⅜ in.	2¼ in.	2⅛ in.	2 in.	1⅞ in.	1¾ in.	1⅝ in.	1½ in.
10.....	0.66	0.69	0.72	0.75	0.79	0.83	0.87	0.92	0.98	1.05	1.12	1.21	1.31
	1.75	**1.83**	**1.91**	**2.00**	**2.10**	**2.21**	**2.33**	**2.47**	**2.62**	**2.80**	**3.00**	**3.23**	**3.50**
11.....	0.60	0.62	0.65	0.68	0.71	0.75	0.79	0.84	0.89	0.95	1.02	1.10	1.19
	1.92	**2.01**	**2.10**	**2.20**	**2.31**	**2.43**	**2.56**	**2.72**	**2.88**	**3.08**	**3.30**	**3.55**	**3.85**
12.....	0.55	0.57	0.59	0.62	0.65	0.69	0.73	0.77	0.82	0.87	0.93	1.01	1.09
	2.10	**2.19**	**2.29**	**2.40**	**2.52**	**2.65**	**2.80**	**2.96**	**3.15**	**3.36**	**3.60**	**3.88**	**4.19**
13.....	0.51	0.53	0.55	0.58	0.61	0.64	0.67	0.71	0.75	0.81	0.87	0.93	1.00
	2.27	**2.38**	**2.48**	**2.60**	**2.73**	**2.88**	**3.03**	**3.21**	**3.41**	**3.64**	**3.90**	**4.20**	**4.54**
14.....	0.47	0.49	0.51	0.53	0.56	0.59	0.63	0.66	0.70	0.75	0.80	0.86	0.93
	2.45	**2.56**	**2.67**	**2.80**	**2.94**	**3.10**	**3.26**	**3.46**	**3.67**	**3.92**	**4.20**	**4.52**	**4.90**
15.....	0.44	0.46	0.48	0.50	0.52	0.55	0.58	0.62	0.66	0.70	0.75	0.81	0.87
	2.62	**2.74**	**2.87**	**3.00**	**3.15**	**3.32**	**3.50**	**3.71**	**3.94**	**4.20**	**4.50**	**4.85**	**5.24**
16.....	0.41	0.43	0.45	0.47	0.49	0.52	0.55	0.58	0.61	0.65	0.70	0.76	0.82
	2.80	**2.92**	**3.05**	**3.20**	**3.36**	**3.54**	**3.73**	**3.95**	**4.20**	**4.48**	**4.80**	**5.17**	**5.60**
17.....	0.38	0.40	0.42	0.44	0.46	0.49	0.52	0.55	0.58	0.62	0.66	0.71	0.77
	2.97	**3.10**	**3.24**	**3.40**	**3.56**	**3.75**	**3.96**	**4.19**	**4.46**	**4.76**	**5.10**	**5.49**	**5.94**
18.....	0.36	0.38	0.40	0.42	0.44	0.46	0.49	0.52	0.55	0.58	0.62	0.67	0.73
	3.15	**3.29**	**3.44**	**3.60**	**3.78**	**3.98**	**4.20**	**4.45**	**4.73**	**5.05**	**5.40**	**5.82**	**6.28**
19.....	0.34	0.36	0.38	0.40	0.42	0.44	0.46	0.49	0.52	0.55	0.59	0.64	0.69
	3.32	**3.47**	**3.62**	**3.80**	**3.98**	**4.20**	**4.43**	**4.69**	**4.98**	**5.32**	**5.70**	**6.14**	**6.64**
20.....	0.33	0.34	0.36	0.38	0.40	0.42	0.44	0.46	0.49	0.52	0.56	0.60	0.65
	3.50	**3.65**	**3.82**	**4.00**	**4.20**	**4.42**	**4.66**	**4.93**	**5.25**	**5.60**	**6.00**	**6.46**	**7.00**
21.....	0.31	0.32	0.34	0.36	0.38	0.40	0.42	0.44	0.47	0.50	0.53	0.57	0.62
	3.67	**3.84**	**4.01**	**4.20**	**4.40**	**4.64**	**4.90**	**5.18**	**5.51**	**5.88**	**6.30**	**6.78**	**7.34**
22.....	0.30	0.31	0.32	0.34	0.36	0.38	0.40	0.42	0.45	0.48	0.51	0.55	0.60
	3.85	**4.02**	**4.20**	**4.40**	**4.62**	**4.86**	**5.13**	**5.43**	**5.77**	**6.16**	**6.60**	**7.11**	**7.70**
23.....	0.29	0.30	0.31	0.32	0.34	0.36	0.38	0.40	0.43	0.46	0.49	0.53	0.57
	4.02	**4.20**	**4.38**	**4.60**	**4.83**	**5.08**	**5.36**	**5.67**	**6.03**	**6.44**	**6.90**	**7.43**	**8.05**
24.....	0.28	0.29	0.30	0.31	0.33	0.35	0.37	0.39	0.41	0.44	0.47	0.51	0.55
	4.20	**4.38**	**4.57**	**4.80**	**5.04**	**5.30**	**5.60**	**5.92**	**6.30**	**6.72**	**7.20**	**7.75**	**8.40**
25.....	0.26	0.27	0.28	0.29	0.31	0.33	0.35	0.37	0.39	0.42	0.45	0.48	0.52
	4.37	**4.57**	**4.78**	**5.00**	**5.25**	**5.53**	**5.83**	**6.17**	**6.56**	**7.00**	**7.50**	**8.07**	**8.75**
26.....	0.25	0.26	0.27	0.28	0.30	0.32	0.34	0.36	0.38	0.40	0.43	0.46	0.50
	4.55	**4.75**	**4.96**	**5.20**	**5.46**	**5.75**	**6.06**	**6.42**	**6.82**	**7.28**	**7.80**	**8.39**	**9.10**
27.....	0.24	0.25	0.26	0.27	0.29	0.30	0.32	0.34	0.36	0.38	0.41	0.44	0.48
	4.72	**4.94**	**5.15**	**5.40**	**5.68**	**5.96**	**6.30**	**6.66**	**7.08**	**7.56**	**8.10**	**8.72**	**9.45**
28.....	0.23	0.24	0.25	0.26	0.27	0.29	0.31	0.33	0.35	0.37	0.40	0.43	0.47
	4.90	**5.11**	**5.34**	**5.60**	**5.88**	**6.19**	**6.53**	**6.91**	**7.34**	**7.84**	**8.40**	**9.04**	**9.80**
29.....	0.23	0.24	0.25	0.26	0.27	0.28	0.30	0.32	0.34	0.36	0.39	0.42	0.45
	5.07	**5.30**	**5.53**	**5.80**	**6.09**	**6.41**	**6.77**	**7.16**	**7.60**	**8.12**	**8.70**	**9.36**	**10.15**

INSTRUCTIONS FOR USE OF TABLES 101 TO 104.—The figures in light face type represent the percentage of spiral reinforcement afforded by a spiral wire of the size heading the table at the pitch shown at the top of each column when placed about a core of the diameter (in inches) appearing at the left. The figures in heavy face type represent the weight per foot of column of the percentage of spiral directly above. This weight includes only the wire. Allowance must be made for the extra wire to finish a spiral unit securely at its ends and for the weight of the three or more spacers. Each ⅞-in. channel spacer adds ¾ pound per foot of spiral. Different manufacturers use different spacers and the weight of this item will vary accordingly.

TABLE 102.—PERCENTAGES AND WEIGHTS OF ⅜-IN. ROUND SPIRALS

Core Dia. Inches	PITCH												
	3 in.	2⅞ in.	2¾ in.	2⅝ in.	2½ in.	2⅜ in.	2¼ in.	2⅛ in.	2 in.	1⅞ in.	1¾ in.	1⅝ in.	1½ in.
15	0.98 5.91	1.02 6.16	1.07 6.44	1.12 6.73	1.18 7.08	1.24 7.46	1.31 7.87	1.39 8.32	1.47 8.85	1.57 9.45	1.68 10.1	1.82 10.9	1.96 11.8
16	0.92 6.30	0.96 6.57	1.00 6.87	1.05 7.18	1.10 7.55	1.16 7.95	1.23 8.39	1.30 8.88	1.38 9.44	1.47 10.1	1.58 10.8	1.70 11.6	1.84 12.6
17	0.87 6.69	0.90 6.98	0.94 7.30	0.99 7.63	1.04 8.02	1.09 8.45	1.15 8.92	1.22 9.44	1.30 10.0	1.39 10.7	1.48 11.5	1.60 12.3	1 73 13.4
18	0.82 7.08	0.85 7.39	0.89 7.73	0.93 8.08	0.98 8.50	1.03 8.94	1.09 9.44	1.16 10.0	1.23 10.6	1.31 11.3	1.40 12.1	1.51 13.1	1.64 14.2
19	0.78 7.48	0.81 7.81	0.84 8.15	0.88 8.53	0.93 8.97	0.98 9.45	1.03 9.97	1.09 10.5	1.16 11.2	1.24 12.0	1.33 12.8	1.43 13.8	1.55 14.9
20	0.74 7 87	0.77 8.21	0.80 8.59	0.84 8.98	0.88 9.44	0.93 9.94	0.98 10.5	1.04 11.1	1.10 11.8	1.18 12.6	1.26 13.5	1.36 14.5	1.47 15.7
21	0.70 8.26	0.73 8.62	0.76 9.02	0.80 9.43	0.84 9.92	0.88 10.4	0.94 11.0	0.99 11.6	1.05 12.4	1.12 13.2	1.20 14.2	1.29 15.2	1.40 16.5
22	0.67 8.66	0.70 9.04	0.73 9.45	0.76 9.88	0.80 10.4	0.84 10.9	0.89 11.5	0.94 12.2	1.00 13.0	1.07 13.8	1.15 14.8	1.23 16.0	1.34 17.3
23	0.64 9.05	0.67 9.45	0.70 9.88	0.73 10.3	0.77 10.9	0.81 11.4	0.85 12.1	0.90 12.8	0.96 13.6	1.02 14.5	1.10 15.5	1.18 16.7	1.28 18.1
24	0.61 9.45	0.64 9.86	0.67 10.3	0.70 10.8	0.73 11.3	0.77 11.9	0.82 12.6	0.87 13.3	0.92 14.2	0.98 15.1	1.05 16.2	1.13 17.4	1.23 18.9
25	0.59 9.84	0.61 10.3	0.64 10.7	0.67 11.2	0.70 11.8	0.74 12.4	0.78 13.1	0.83 13.9	0.88 14.7	0.94 15.7	1.01 16.9	1.09 18.2	1.18 19.7
26	0.57 10.2	0.59 10.7	0.62 11.2	0.65 11.7	0.68 12.3	0.71 12.9	0.75 13.6	0.80 14.4	0.85 15.4	0.90 16.4	0.97 17.5	1.05 18.9	1.13 20.4
27	0.55 10.6	0.57 11.1	0.59 11.6	0.62 12.1	0.65 12.7	0.69 13.4	0.73 14.2	0.77 15.0	0.82 15.9	0.87 17.0	0.94 18.2	1.01 19.6	1.09 21.2
28	0.53 11.0	0.55 11.5	0.57 12.0	0.60 12.6	0.63 13.2	0.66 13.9	0.70 14.7	0.74 15.5	0.79 16.5	0.84 17.6	0.90 18.9	0.97 20.4	1.05 22.0
29	0.51 11.4	0.53 11.9	0.55 12.5	0.58 13.0	0.61 13.7	0.64 14.4	0.68 15.2	0.72 16.1	0.76 17.1	0.81 18.3	0.87 19.6	0.94 21.1	1.01 22.8
30	0.49 11.8	0.51 12.3	0.53 12.9	0.56 13.5	0.59 14.2	0.62 14.9	0.65 15.7	0.69 16.6	0.74 17.7	0.79 18.9	0.84 20.2	0.91 21.8	0.98 23.6
31	0.48 12.2	0.50 12.7	0.52 13.3	0.54 13.9	0.57 14.6	0.60 15.4	0.63 16.3	0 67 17.2	0.72 18.3	0.76 19.5	0.82 20.9	0.88 22.5	0.95 24.4
32	0.46 12.6	0.48 13.1	0.50 13.7	0.52 14.4	0.55 15.1	0.58 15.9	0.61 16.8	0.65 17.8	0.69 18.9	0.74 20.1	0.79 21.6	0.85 23.3	0.92 25.2
33	0.45 13.0	0.47 13.5	0.49 14.2	0.51 14.8	0.54 15.6	0.57 16.4	0.60 17.3	0.63 18.3	0.67 19.5	0.71 20.8	0.77 22.3	0.83 24.0	0.89 26.0
34	0.44 13.4	0.45 14.0	0.47 14.6	0.49 15.3	0.52 16.1	0.55 16.9	0.58 17.8	0.61 18.9	0.65 20.1	0.70 21.4	0.75 23.0	0.81 24.7	0.87 26.8
35	0.42 13.8	0.44 14.4	0.46 15.0	0.48 15.7	0.50 16.5	0.53 17.4	0.56 18.4	0.59 19.4	0.63 20.7	0.67 22.0	0.72 23.6	0.78 25.4	0.84 27.5
36	0.41 14.2	0.43 14.8	0.45 15.4	0.47 16.2	0.49 17.0	0.52 17.9	0.55 18.9	0.58 20.0	0.61 21.3	0.65 22.7	0.70 24.3	0.75 26.2	0.82 28.3
37	0.40 14.6	0.41 15.2	0.43 15.9	0.45 16.6	0.47 17.5	0.50 18.4	0.53 19.4	0.56 20.6	0.60 21.9	0.64 23.3	0.68 25.0	0.73 26.9	0.80 29.1
38	0.39 15.0	0.40 15.6	0.42 16.3	0.44 17.1	0.46 18.0	0.49 18.9	0.52 19.9	0.55 21.1	0.58 22.4	0.62 23.9	0.66 25.6	0.71 27.6	0.77 29.9
39	0.38 15.4	0.39 16.0	0.41 16.7	0.43 17.5	0.45 18.4	0.47 19.4	0.50 20.5	0.53 21.6	0.57 23.0	0.61 24.6	0.65 26.3	0.70 28.3	0.76 30.7

See instructions for use under Table 101.

TABLE 103.—PERCENTAGES AND WEIGHTS OF ½-IN. ROUND SPIRALS

Core Dia. Inches	PITCH												
	3 in.	2⅞ in.	2¾ in.	2⅝ in.	2½ in.	2⅜ in.	2¼ in.	2⅛ in.	2 in.	1⅞ in.	1¾ in.	1⅝ in.	1½ in.
15.....	1.75	1.82	1.90	1.99	2.09	2.21							
	10.5	11.0	11.4	12.0	12.6	13.2							
16.....	1.63	1.70	1.78	1.87	1.96	2.07							
	11.2	11.7	12.2	12.8	13.4	14.1							
17.....	1.54	1.61	1.68	1.76	1.85	1.95	2.05						
	11.9	12.4	13.0	13.6	14.3	15.0	15.9						
18.....	1.45	1.52	1.59	1.66	1.75	1.84	1.94	2.06					
	12.6	13.1	13.7	14.4	15.1	15.9	16.8	17.8					
19.....	1.38	1.44	1.50	1.57	1.65	1.74	1.84	1.95	2.07				
	13.3	13.9	14.5	15.2	16.0	16.8	17.7	18.8	20.0				
20.....	1.30	1.36	1.43	1.50	1.57	1.65	1.74	1.84	1.96	2.09			
	14.0	14.6	15.3	16.0	16.8	17.7	18.6	19.8	21.0	22.4			
21.....	1.25	1.30	1.36	1.43	1.50	1.57	1.66	1.76	1.87	2.00			
	14.7	15.3	16.0	16.8	17.6	18.5	19.6	20.8	22.0	23.5			
22.....	1.19	1.24	1.30	1.36	1.43	1.50	1.59	1.68	1.79	1.90	2.04		
	15.4	16.1	16.8	17.6	18.5	19.4	20.5	21.7	23.1	24.6	26.4		
23.....	1.14	1.19	1.24	1.30	1.37	1.44	1.52	1.60	1.71	1.82	1.95	2.10	
	16.1	16.8	17.6	18.4	19.3	20.3	21.4	22.7	24.1	25.8	27.6	29.7	
24.....	1.09	1.14	1.19	1.25	1.31	1.38	1.45	1.54	1.64	1.75	1.87	2.02	
	16.8	17.5	18.3	19.2	20.2	21.2	22.4	23.7	25.2	26.9	28.8	31.0	
25.....	1.05	1.09	1.14	1.20	1.26	1.32	1.40	1.48	1.57	1.67	1.80	1.94	2.10
	17.5	18.3	19.1	20.0	21.0	22.1	23.3	24.7	26.2	28.0	30.0	32.3	35.0
26.....	1.01	1.05	1.10	1.15	1.21	1.27	1.34	1.42	1.51	1.60	1.73	1.86	2.01
	18.2	19.0	19.9	20.8	21.8	23.0	24.3	25.7	27.2	29.1	31.2	33.6	36.4
27.....	0.97	1.01	1.06	1.11	1.16	1.22	1.29	1.37	1.45	1.55	1.66	1.78	1.94
	18.9	19.7	20.6	21.6	22.7	23.9	25.2	26.7	28.3	30.2	32.4	34.9	37.8
28.....	0.93	0.97	1.02	1.07	1.12	1.18	1.25	1.32	1.40	1.49	1.60	1.72	1.87
	19.6	20.4	21.4	22.4	23.5	24.7	26.1	27.7	29.4	31.3	33.6	36.2	39.2
29.....	0.90	0.94	0.98	1.03	1.08	1.14	1.20	1.27	1.35	1.44	1.54	1.66	1.80
	20.3	21.2	22.2	23.2	24.4	25.6	27.1	28.7	30.4	32.5	34.8	37.5	40.6
30.....	0.87	0.91	0.95	1.00	1.05	1.10	1.16	1.23	1.31	1.40	1.50	1.61	1.74
	21.0	21.9	22.9	24.0	25.2	26.5	28.0	29.6	31.5	33.6	36.0	38.8	42.0
31.....	0.84	0.88	0.92	0.96	1.01	1.07	1.13	1.19	1.27	1.35	1.45	1.56	1.68
	21.7	22.6	23.6	24.8	26.0	27.4	28.9	30.6	32.5	34.7	37.2	40.0	43.3
32.....	0.82	0.85	0.89	0.93	0.98	1.03	1.09	1.16	1.23	1.31	1.41	1.51	1.64
	22.4	23.4	24.4	25.6	26.8	28.3	29.8	31.6	33.6	35.8	38.4	41.3	44.7
33.....	0.79	0.83	0.87	0.91	0.95	1.00	1.06	1.12	1.19	1.27	1.36	1.46	1.59
	23.1	24.1	25.2	26.4	27.7	29.2	30.8	32.6	34.6	36.9	39.6	42.6	46.1
34.....	0.77	0.80	0.84	0.88	0.92	0.97	1.03	1.09	1.16	1.24	1.32	1.42	1.54
	23.8	24.8	25.9	27.2	28.5	30.0	31.7	33.5	35.6	38.0	40.8	43.9	47.6
35.....	0.75	0.78	0.81	0.85	0.90	0.95	1.00	1.05	1.12	1.19	1.28	1.38	1.50
	24.5	25.6	26.7	28.0	29.4	30.9	32.6	34.5	36.7	39.2	42.0	45.2	48.9
36.....	0.73	0.76	0.79	0.83	0.87	0.92	0.97	1.02	1.09	1.16	1.25	1.35	1.45
	25.2	26.3	27.5	28.8	30.2	31.8	33.6	35.5	37.8	40.3	43.2	46.5	50.4
37.....	0.71	0.74	0.77	0.81	0.85	0.89	0.94	1.00	1.06	1.13	1.21	1.30	1.41
	25.9	27.0	28.2	29.6	31.1	32.7	34.5	36.5	38.8	41.4	44.4	47.8	51.8
38.....	0.69	0.72	0.75	0.79	0.83	0.87	0.92	0.97	1.03	1.10	1.18	1.27	1.38
	26.6	27.8	29.1	30.4	31.9	33.6	35.5	37.5	39.9	42.5	45.6	49.1	53.2
39.....	0.67	0.70	0.73	0.77	0.81	0.85	0.90	0.95	1.01	1.07	1.15	1.24	1.34
	27.3	28.5	29.7	31.2	32.8	34.5	36.4	38.5	40.9	43.6	46.8	50.4	54.6

See instructions for use under Table 101.

TABLE 104.—PERCENTAGES AND WEIGHTS OF ⅝-IN. ROUND SPIRALS

Core Dia. Inches	PITCH												
	3 in.	2⅞ in.	2¾ in.	2⅝ in.	2½ in.	2⅜ in.	2¼ in.	2⅛ in.	2 in.	1⅞ in.	1¾ in.	1⅝ in.	1½ in.
30.....	1.36	1.42	1.49	1.56	1.64	1.72	1.82	1.93	2.05				
	32.8	34.3	35.8	37.5	39.4	41.5	43.8	46.4	49.3				
31.....	1.32	1.38	1.44	1.51	1.59	1.67	1.76	1.87	1.98				
	33.9	35.4	37.0	38.8	40.8	42.9	45.3	47.9	50.9				
32.....	1.28	1.33	1.39	1.46	1.54	1.62	1.71	1.81	1.92	2.04			
	35.1	36.5	38.2	40.1	42.2	44.3	46.7	49.5	52.6	56.1			
33.....	1.24	1.30	1.35	1.42	1.49	1.57	1.66	1.75	1.86	1.99			
	36.1	37.7	39.4	41.3	43.3	45.6	48.2	51.1	54.2	57.8			
34.....	1.21	1.26	1.32	1.38	1.45	1.52	1.61	1.70	1.81	1.93			
	37.3	38.8	40.6	42.6	44.7	47.1	49.7	52.6	55.9	59.6			
35.....	1.17	1.22	1.28	1.34	1.41	1.48	1.56	1.65	1.76	1.87	2.01		
	38.3	40.0	41.8	43.8	46.1	48.4	51.2	54.2	57.5	61.3	65.7		
36.....	1.14	1.19	1.24	1.30	1.37	1.44	1.52	1.61	1.71	1.82	1.95		
	39.4	41.2	42.9	45.0	47.4	49.8	52.6	55.7	59.2	63 1	67.6		
37.....	1.11	1.16	1.21	1.27	1.33	1.40	1.48	1.56	1.66	1.77	1.90	2.04	
	40.5	42.3	44.2	46.3	48.6	51.2	54.1	57.2	60.8	64.8	69.5	74.8	
38.....	1.08	1.13	1.18	1.23	1.29	1.36	1.44	1.52	1.62	1.73	1.85	1.99	
	41.6	43.4	45.3	47.6	49.9	52.6	55.5	58.8	62.4	66.7	71.3	76.8	
39.....	1.05	1.10	1.15	1.20	1.26	1.33	1.40	1.48	1.58	1.68	1.80	1.94	
	42.7	44.6	46.6	48.8	51.2	53.9	56.9	60.3	64.1	68.3	73.2	78.8	
40.....	1.02	1.07	1.12	1.17	1.23	1.29	1.36	1.45	1.54	1.64	1.76	1.89	2.05
	43.8	45.7	47.8	50.1	52.6	55.3	58.4	61.9	65.8	70.1	75.2	80.8	87.6
41.....	1.00	1.04	1.09	1.14	1.20	1.26	1.33	1.41	1.50	1.60	1.71	1.85	2.00
	44.9	46.8	48.9	51.3	53.8	56.7	59.9	63.3	67.4	71.8	76.9	83.8	89.8
42.....	0.98	1.02	1.06	1.12	1.17	1.23	1.30	1.38	1.46	1.56	1.67	1.80	1.95
	46.0	47.9	49.9	52.6	55.2	58.1	61.3	65.0	69.1	73.6	78.8	84.8	92.0
43.....	0.95	1.00	1.04	1.09	1.14	1.20	1.27	1.35	1.43	1.53	1.63	1.76	1.91
	47.0	49.2	51.3	53.8	56.5	59.4	62.8	66.5	70.7	75.3	80.7	86.9	94.2
44.....	0.93	0.98	1.02	1.07	1.12	1.18	1.24	1.32	1.40	1.49	1.60	1.72	1.87
	48.2	50.3	52.5	55.1	57.8	60.9	64.2	68.2	72.3	77.1	82.6	88.9	96.5
45.....	0.91	0.95	0.99	1.04	1.09	1.15	1.21	1.29	1.37	1.46	1.56	1.68	1.82
	49.2	51.4	53.7	56.3	59.1	62.2	65.7	69.6	74.0	78.9	84.5	90.9	98.6
46.....	0.89	0.93	0.97	1.02	1.07	1.13	1.19	1.26	1.34	1.43	1.53	1.65	1.78
	50.3	52.6	54.8	57.6	60.4	63.7	67.2	71.2	75.7	80.7	86.4	93.0	100.7
47.....	0.87	0.91	0.95	1.00	1.05	1.10	1.16	1.23	1.31	1.40	1.50	1.61	1.74
	51.4	53.7	56.1	58.8	61.8	65.0	68.7	72.7	77.3	82.4	88.3	95.0	102.8
48.....	0.85	0.89	0.93	0.98	1.02	1.08	1.14	1.20	1.28	1.36	1.46	1.57	1.71
	52.6	54.8	57.3	60.1	63.1	66.4	70.1	74.2	78.9	84.1	90.1	97.0	105.1
49.....	0.84	0.87	0.91	0.96	1.00	1.06	1.12	1.18	1.25	1.34	1.44	1.54	1.68
	53.7	55.9	58.4	61.3	64.4	67.8	71.6	75.7	80.6	85.9	92.0	99.1	107.3

See instructions for use under Table 101.

For spirals of heavier wire or larger diameter than are included in Tables 101 to 104, the percentage may computed approximately by the formula:

$$p' = \frac{4as}{(d)\ (\text{pitch})}$$

in which p' = percentage of area of circle of diameter, d.
as = cross sectional area of spiral rod.
d = diameter of spiral (generally listed as overall or outside diameter).
pitch = distance between centers of successive turns measured parallel to axis of spiral.

TABLE 105.—PERIMETERS, VOLUMES AND CORE AREAS FOR ROUND COLUMNS

Diameter		Core Area, sq. in.	Rd. Column Perimeter		Volume, cu. ft. per ft.		
Column, in.	Core, in.		Ft.	In.	Round	Octagonal	Square
14	10	78.5	3	8	1.07	1.13	1.36
	11	95.0					
16	12	113.1	4	2	1.40	1.47	1.78
	13	132.7					
18	14	153.9	4	9	1.77	1.86	2.25
	15	176.7					
20	16	201.0	5	3	2.18	2.30	2.78
	17	227.0					
22	18	254.5	5	9	2.64	2.78	3.36
	19	283.5					
24	20	314.2	6	3	3.14	3.31	4.00
	21	346.4					
26	22	380.1	6	10	3.69	3.89	4.69
	23	415.4					
28	24	452.4	7	4	4.28	4.51	5.44
	25	490.9					
30	26	530.9	7	10	4.91	5.18	6.25
	27	572.6					
32	28	615.8	8	5	5.58	5.89	7.12
	29	660.5					
34	30	706.9	8	11	6.30	6.55	8.02
	31	754.8					
36	32	804.2	9	5	7.07	7.46	9.00
	33	855.3					
38	34	907.9	10	0	7.88	8.31	10.03
	35	962.1					
40	36	1,017.9	10	6	8.73	9.21	11.11
	37	1,075.2					

TABLE 106.—VOLUMES OF FLAT SLAB COLUMN SHAFTS AND CAPITALS

Round Columns								Square Columns							
Diameter of Column Shaft	Volume of Column Shaft	Diameter of Column Capital						Side of Column Shaft	Volume of Column Shaft	Side of Column Capital					
		3 ft. 6 in.	4 ft. 0 in.	4 ft. 6 in.	5 ft. 0 in.	5 ft. 6 in.	6 ft. 0 in.			3 ft. 6 in.	4 ft. 0 in.	4 ft. 6 in.	5 ft. 0 in.	5 ft. 6 in.	6 ft. 0 in.
14	1.07	5.23	....	....	....	....	....	14	1.36	6.65	....	....	....	....	....
16	1.40	4.81	7.61	....	....	....	....	16	1.78	6.13	9.67	....	....	....	....
18	1.77	4.38	7.07	10.58	....	....	....	18	2.25	5.59	9.01	13.49	....	....	....
20	2.18	3.93	6.52	9.94	14.30	....	....	20	2.78	5.01	8.30	12.66	18.21	....	....
22	2.64	3.48	5.95	9.25	13.48	18.75	....	22	3.36	4.43	7.57	11.79	17.18	23.90	....
24	3.14	3.02	5.36	8.54	12.66	17.80	24.08	24	4.00	3.85	6.83	10.88	16.12	22.68	30.67
26	3.69	2.55	4.78	7.83	11.80	16.82	22.96	26	4.69	3.26	6.09	9.96	15.03	21.41	29.23
28	4.28	2.13	4.18	7.08	10.91	15.78	21.76	28	5.44	2.71	5.33	9.03	13.90	20.09	27.71
30	4.91	1.70	3.60	6.34	10.00	14.72	20.54	30	6.25	2.16	4.59	8.08	12.75	18.75	26.17
32	5.58	....	3.04	5.60	9.12	13.66	19.31	32	7 11	....	3.87	7.14	11.61	17.40	24.60
34	6.30	....	2.48	4.89	8.21	12.58	18.05	34	8.03	....	3.18	6.23	10.46	16.01	23.00
36	7.07	....	2.01	4.20	7.32	11.50	16.78	36	9.00	....	2.51	5.35	9.32	14.66	21.38
38	7.88	....	....	3.53	6.44	10.40	15.51	38	10.02	....	....	4.50	8.20	13.27	19.76
40	8.73	....	....	2.88	5.60	9.36	14.24	40	11.11	....	....	3.67	7.13	11.91	18.13
42	9.62	....	....	2.26	4.78	8.31	12.96	42	12.25	....	....	2.90	6.09	10.59	16.51
44	10.56	....	....	....	4.00	7.29	11.71	44	13.44	....	....	....	5.09	9.29	14.92
46	11.54	....	....	....	3.24	6.31	10.48	46	14.69	....	....	....	4.14	8.04	13.35
48	12.57	....	....	....	2.55	5.36	9.29	48	16.00	....	....	....	3.25	6.84	11.82

See note under Table 107, next page.

TABLE 107.—AREAS AND PERIMETERS OF ROUND SECTIONS

Diameter		Area		Perimeter	Diameter		Area		Perimeter
Ft. In.	In.	Sq. In.	Sq. Ft.	Ft.	Ft. In.	In.	Sq. In.	Sq. Ft.	Ft.
1-1	13	132.7	0.92	3.40	4-7	55	2,376	16.50	14.40
1-2	14	153.9	1.07	3.66	4-8	56	2,463	17.10	14.66
1-3	15	176.7	1.23	3.93	4-9	57	2,552	17.72	14.92
1-4	16	201.1	1.40	4.18	4-10	58	2,642	18.35	15.18
1-5	17	227.0	1.58	4.45	4-11	59	2,734	18.99	15.45
1-6	18	254.5	1.77	4.71	5-0	60	2,827	19.63	15.71
1-7	19	283.5	1.97	4.97	5-1	61	2,922	20.29	15.97
1-8	20	314.2	2.18	5.23	5-2	62	3,019	20.97	16.23
1-9	21	346.4	2.41	5.50	5-3	63	3,117	21.65	16.49
1-10	22	380.1	2.64	5.76	5-4	64	3,217	22.34	16.76
1-11	23	415.5	2.89	6.02	5-5	65	3,318	23.04	17.02
2-0	24	452.4	3.14	6.28	5-6	66	3,421	23.76	17.28
2-1	25	490.9	3.41	6.55	5-7	67	3,526	24.48	17.54
2-2	26	530.9	3.69	6.81	5-8	68	3,632	25.22	17.80
2-3	27	572.6	3.98	7.07	5-9	69	3,739	25.97	18.06
2-4	28	615.8	4.28	7.33	5-10	70	3,848	26.72	18.32
2-5	29	660.5	4.59	7.58	5-11	71	3,959	27.49	18.58
2-6	30	706.9	4.91	7.86	6-0	72	4,072	28.27	18.85
2-7	31	754.8	5.24	8.12	6-1	73	4,185	29.06	19.11
2-8	32	804.2	5.58	8.38	6-2	74	4,301	29.87	19.37
2-9	33	855.3	5.94	8.64	6-3	75	4,418	30.68	19.63
2-10	34	907.9	6.30	8.89	6-4	76	4,536	31.50	19.90
2-11	35	962.1	6.68	9.16	6-5	77	4,657	32.34	20.16
3-0	36	1,017.9	7.07	9.42	6-6	78	4,778	33.18	20.42
3-1	37	1,075.2	7.47	9.68	6-7	79	4,902	34.04	20.68
3-2	38	1,134.1	7.88	9.95	6-8	80	5,027	34.91	20.94
3-3	39	1,194.6	8.30	10.21	6-9	81	5,153	35.78	21.21
3-4	40	1,256.6	8.73	10.47	6-10	82	5,281	36.67	21.47
3-5	41	1,320.3	9.17	10.72	6-11	83	5,411	37.58	21.73
3-6	42	1,385.4	9.62	10.99	7-0	84	5,542	38.48	21.99
3-7	43	1,452.2	10.08	11.26	7-1	85	5,674	39.40	22.25
3-8	44	1,520.5	10.56	11.52	7-2	86	5,809	40.34	22.51
3-9	45	1,590.4	11.04	11.79	7-3	87	5,945	41.28	22.78
3-10	46	1,661.9	11.54	12.04	7-4	88	6,082	42.24	23.04
3-11	47	1,734.9	12.05	12.31	7-5	89	6,221	43.20	23.30
4-0	48	1,809.6	12.57	12.57	7-6	90	6,362	44.18	23.56
4-1	49	1,885.7	13.10	12.83	7-7	91	6,504	45.17	23.82
4-2	50	1,963.5	13.64	13.10	7-8	92	6,648	46.16	24.09
4-3	51	2,042.8	14.19	13.35	7-9	93	6,793	47.17	24.35
4-4	52	2,123.7	14.75	13.61	7-10	94	6,940	48.19	24.61
4-5	53	2,206.2	15.32	13.88	7-11	95	7,088	49.22	24.87
4-6	54	2,290.2	15.90	14.14	8-0	96	7,238	50.27	25.13

INSTRUCTIONS FOR USE OF TABLE 106. For octagonal columns and capitals add 5½ per cent to values given for round columns and capitals. Volume of concrete in column shafts is given in cubic feet per foot of height. For column capitals it is given in cubic feet and includes only the concrete in the capital outside the surface of the column enclosed in the capital.

PROPORTIONS ADOPTED FOR DESIGN DIAGRAMS FOR SQUARE FOOTINGS.

$$d_v = (b)\left(\frac{d}{b}\right)\frac{0.491-\left(\frac{d}{b}\right)}{0.466}$$

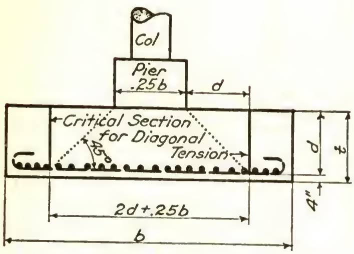


FIG. 16.—FLAT-TOP FOOTINGS.

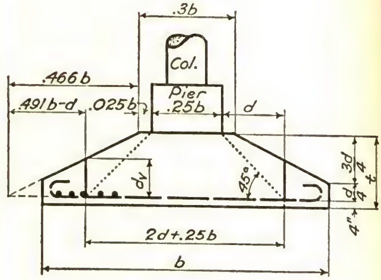


FIG. 17.—SLOPED-TOP FOOTINGS.

INSTRUCTIONS FOR USE OF DIAGRAM 108

This diagram applies only to footings of the standardized proportions shown in Figs. 16 and 17. If the footing bars are straight and extend to within 2 inches of the edges of the footing the value of v_c is 0.02 $f'c$. If the footing bars are provided with special anchorage the total length of the hooked bar being the overall width of the footing plus 20 bar diameters, the value of v_c may be taken as 0.03 $f'c$. The value of w is the load at the top of the footing divided by the area of the base of the footing. d is the effective depth of footing to the reinforcing steel and b the side of the square base of the footing.

INSTRUCTIONS FOR USE OF DIAGRAM 109

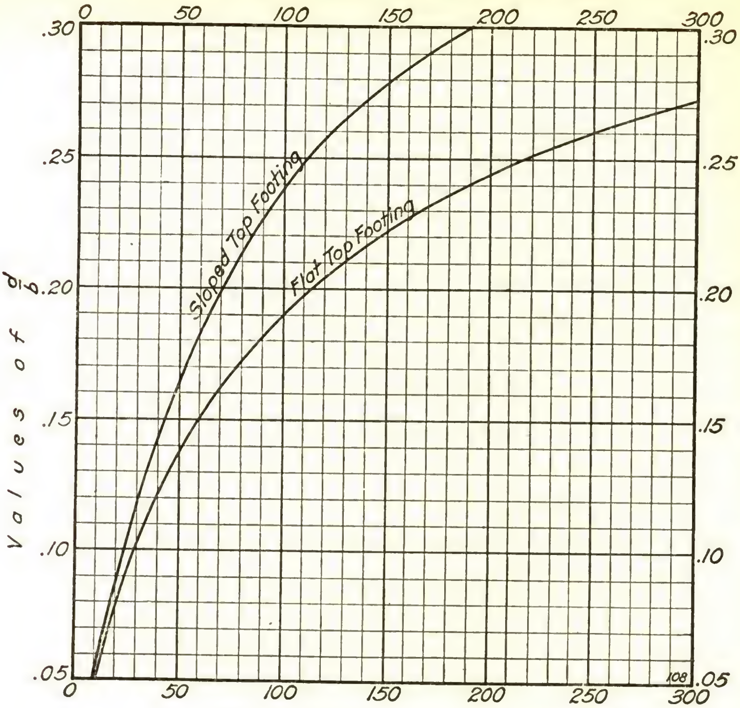
This diagram is based on special anchorage of footing bars where higher shearing stresses (0.03 $f'c$) are used in the design. It applies only to square footings, either sloped or flat topped, of the standardized proportions shown in Figs. 16 and 17. The *length* of footing bars must be equal to the overall width of the base of the footing plus twenty bar diameters.

To determine the maximum bar size enter the diagram with the side of the footing, and proceed vertically to the sloping index line marked with the strength of the concrete (2000 to 3750-lb.) and the kind of bar (plain or deformed) to be used. From this intersection pass horizontally to the marginal scales and use a bar not larger than the size at or next below this point on the scale.

INSTRUCTIONS FOR USE OF DIAGRAMS 113, 115, 117, ETC., TO 143

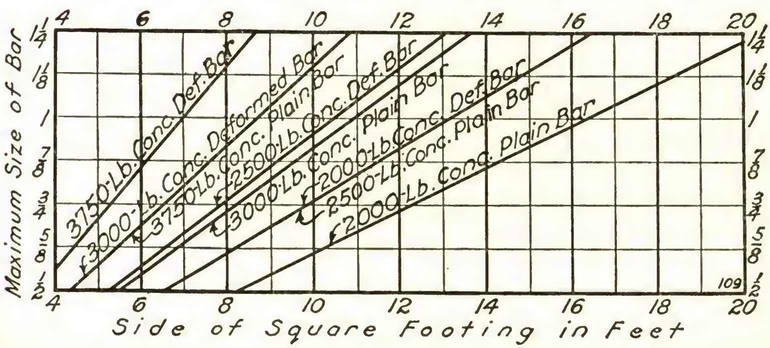
Enter the diagram at the top with the load to be applied at the top of the footing. For small piers this will be the same as the basement story column load. Proceed vertically to an intersection with that one of the upper index lines marked with the strength of the concrete to be used in the design and read off the volume of concrete required for one footing (without pier). Proceed vertically to an intersection with the middle group of index lines and here read off the square feet of formwork required for one footing (exclusive of the pier forms). No forms are included for sloping tops of footings, as the concrete for such footings should be made stiff enough to place without top forms. Continue vertically to an intersection with the lower group of index lines and read off the weight of reinforcing steel required for one footing. No column dowel bars are included and no stirrups or other web reinforcement are necessary. Every bar is bent with a hook at each end.

DIAGRAM 108.—EFFECTIVE DEPTH OF FOOTINGS



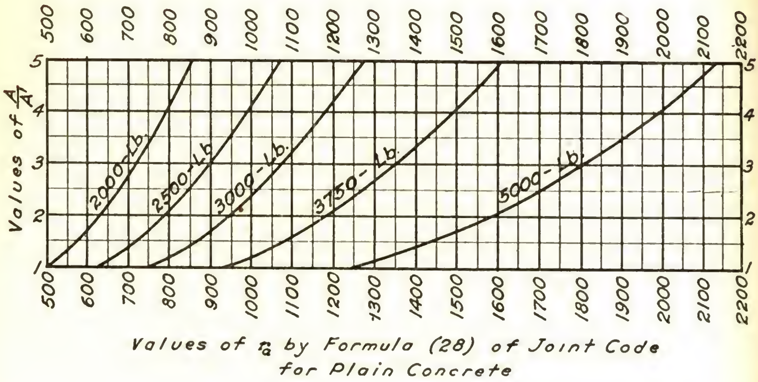
See instructions for use under Fig. 16, page 148.

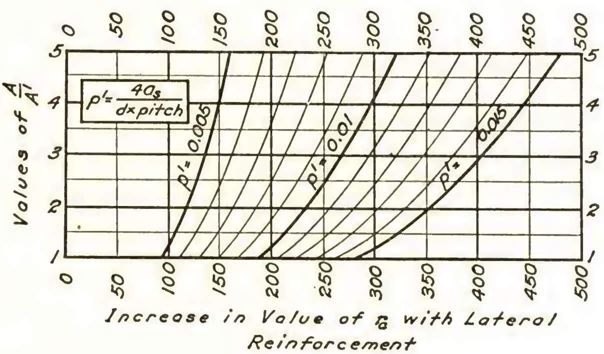
DIAGRAM 109.—MAXIMUM BAR SIZE IN FOOTINGS



See instructions for use under Fig. 16, page 148.

DIAGRAM 110.—TRANSFER OF COLUMN LOAD TO TOP OF PIER



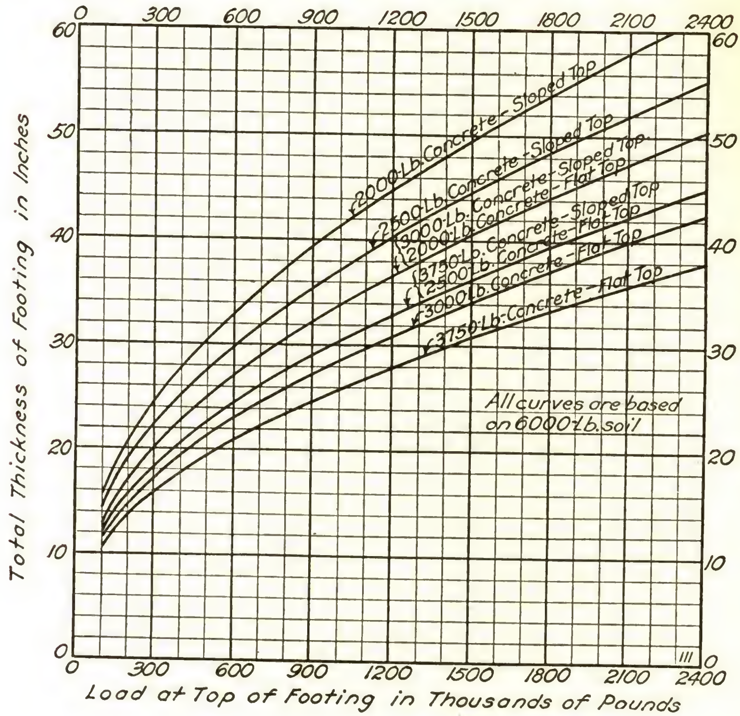


INSTRUCTIONS FOR USE.—In this diagram
A is the area of the top of pier or footing.
A' is the loaded area of the top of pier or footing that is the area of the column section used in design.
ra is the permissible unit load on the loaded area.

When using the increased value of ra (on account of spiral reinforcement in the pier) as given in the lower part of the diagram, the value of A must be taken as the area of the top of the pier within the outside diameter of the spiral and this value of A must be used in both the upper and lower portions of the diagram and the two values of ra added, to get the total permissible unit load where a spiral is used. See instructions under Table 104 for definition of symbols used in formula for p'.

The column vertical bars, or an equal steel area in the form of dowels, must extend at least 24 bar diameters for deformed bars or 30 bar diameters for plain bars above and below the base of column.

DIAGRAM 111.—TOTAL THICKNESS OF FOOTINGS WHEN $v_c = 0.03 f'_c$



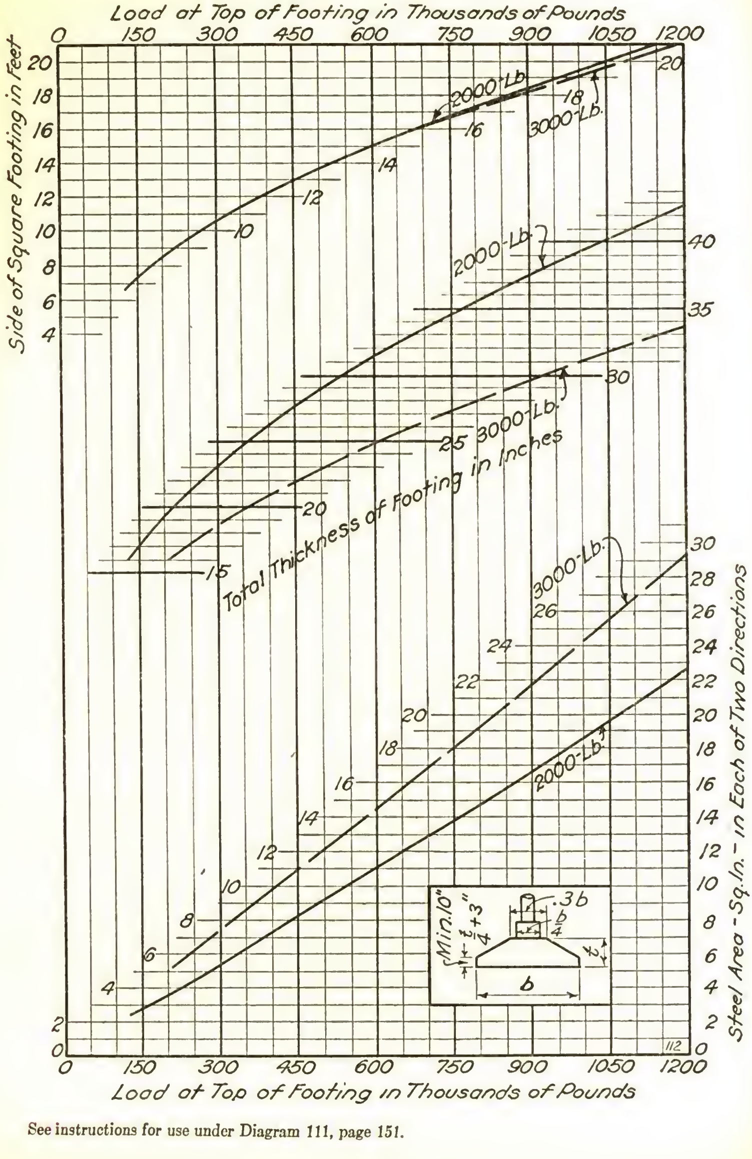
INSTRUCTIONS FOR USE.—This diagram gives an *approximate* indication of the thickness of footing required when the footing bars are anchored as described under Fig. 16. Flat-top footings are based on the standardized proportions shown in Fig. 16 and sloped-top footings on those of Fig. 17. For soil pressures less than 6000-lb. per sq. ft. the total thickness of footings will be slightly less than the values taken from this diagram.

INSTRUCTIONS FOR USE OF DIAGRAMS 112, 114, 116, ETC , TO 142.

Enter this diagram at the top with the load applied at the top of the footing. For small piers this will be the same as the basement column load. Proceed vertically to an intersection with that one of the upper index lines marked with the strength of concrete to be used in the design and read off the dimension in feet of the side of the square base of the footing. Continue vertically to an intersection with the middle group of index lines and here read off the total thickness of the footing in inches. Continue vertically to an intersection with the lower group of index lines and read off the area in sq. in. of the bars required in each of two directions. Use Diagram 109 to determine maximum size of footing bar.

DESIGN OF SLOPED-TOP FOOTINGS WITH $v_c = 0.03 f'_c$

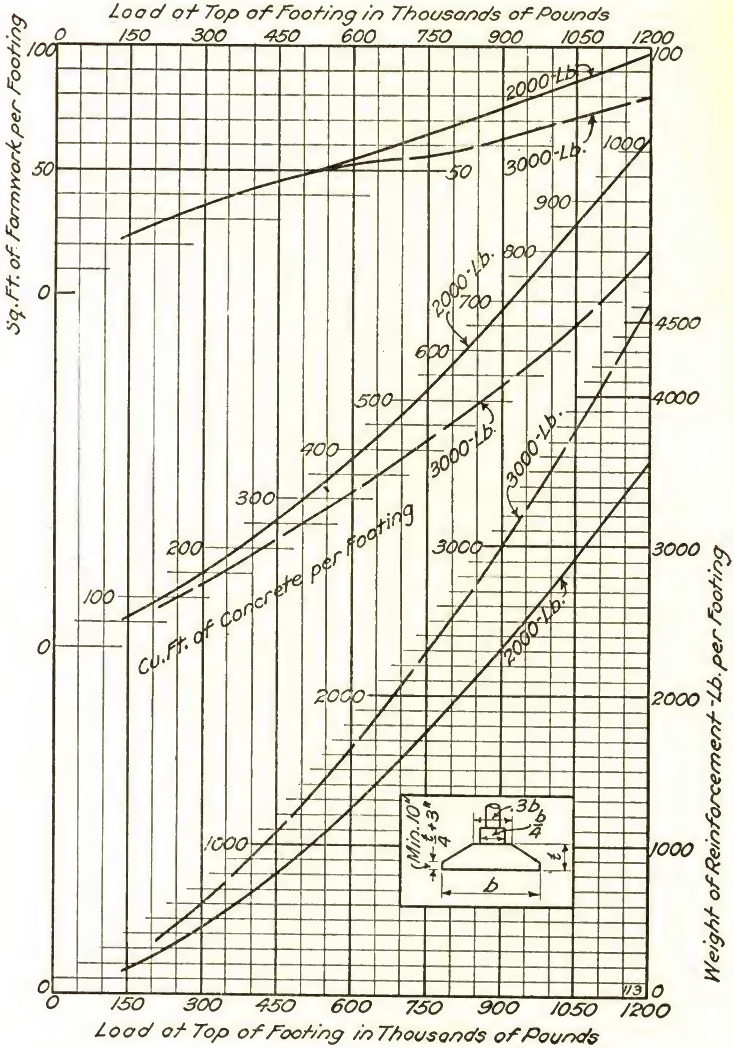
DIAGRAM 112.—3000-LB. SOIL — 2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Diagram 111, page 151.

QUANTITIES FOR SLOPED-TOP FOOTINGS WITH $v_c = 0.03f'_c$

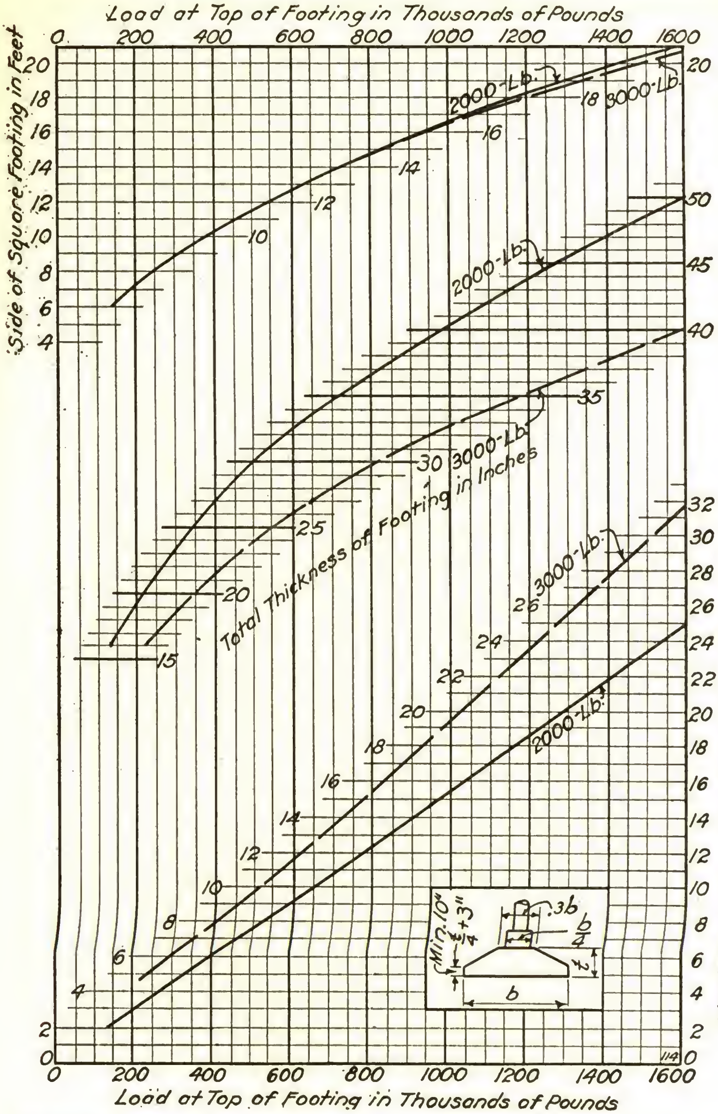
DIAGRAM 113.—3000-LB. SOIL—2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Fig. 16, page 148.

DESIGN OF SLOPED-TOP FOOTINGS WITH $v_c = 0.03 f'_c$

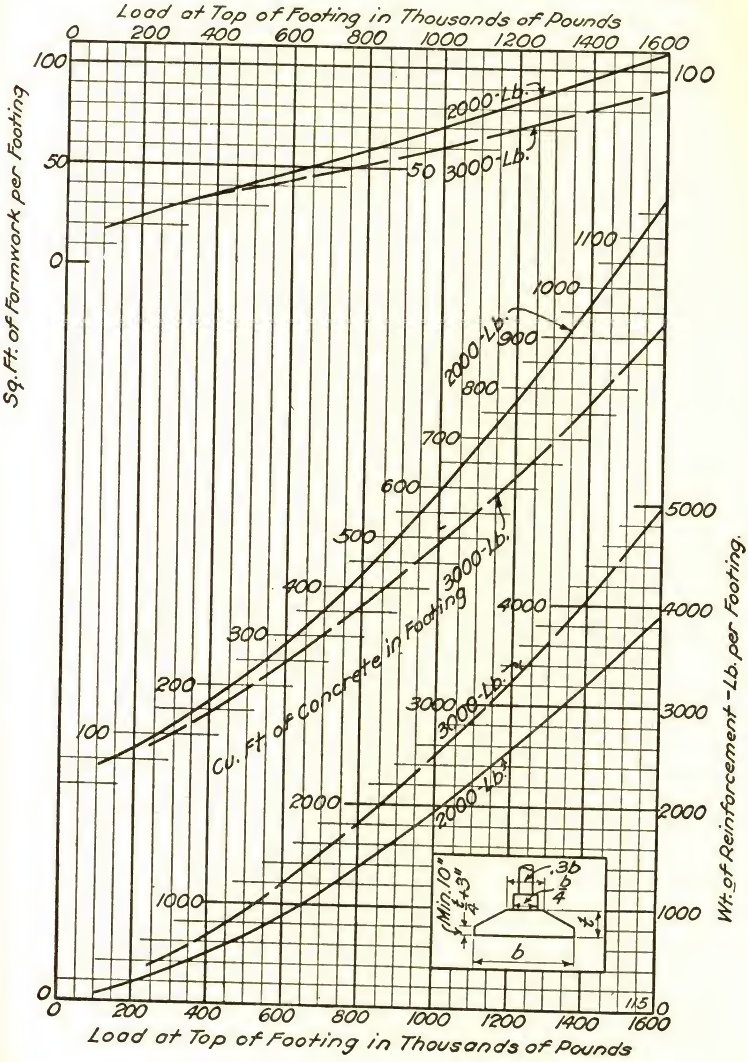
DIAGRAM 114.—4000-LB. SOIL — 2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Diagram 111, page 151.

QUANTITIES FOR SLOPED-TOP FOOTINGS WITH $v_c = 0.03f'_c$

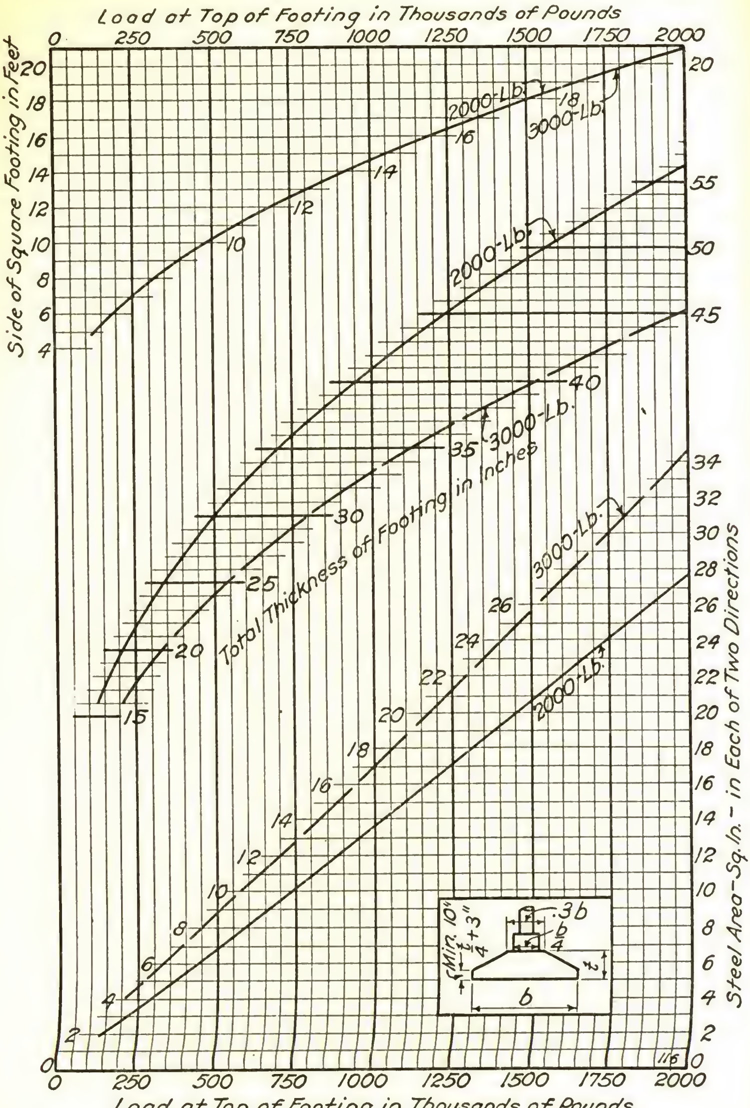
DIAGRAM 115.—4000-LB. SOIL—2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Fig. 16, page 148.

DESIGN OF SLOPED-TOP FOOTINGS WITH $v_c = 0.03 f'_c$

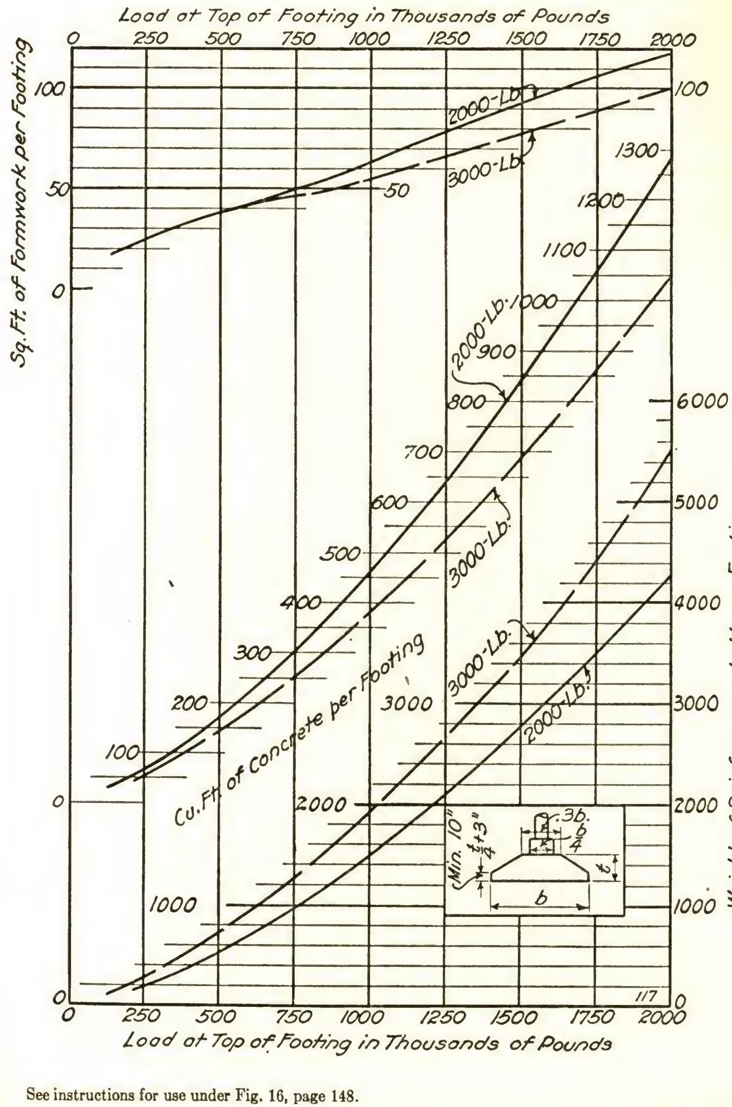
DIAGRAM 116.—5000-LB. SOIL — 2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Diagram 111, page 151.

QUANTITIES FOR SLOPED-TOP FOOTINGS WITH $v_c = 0.03f'_c$

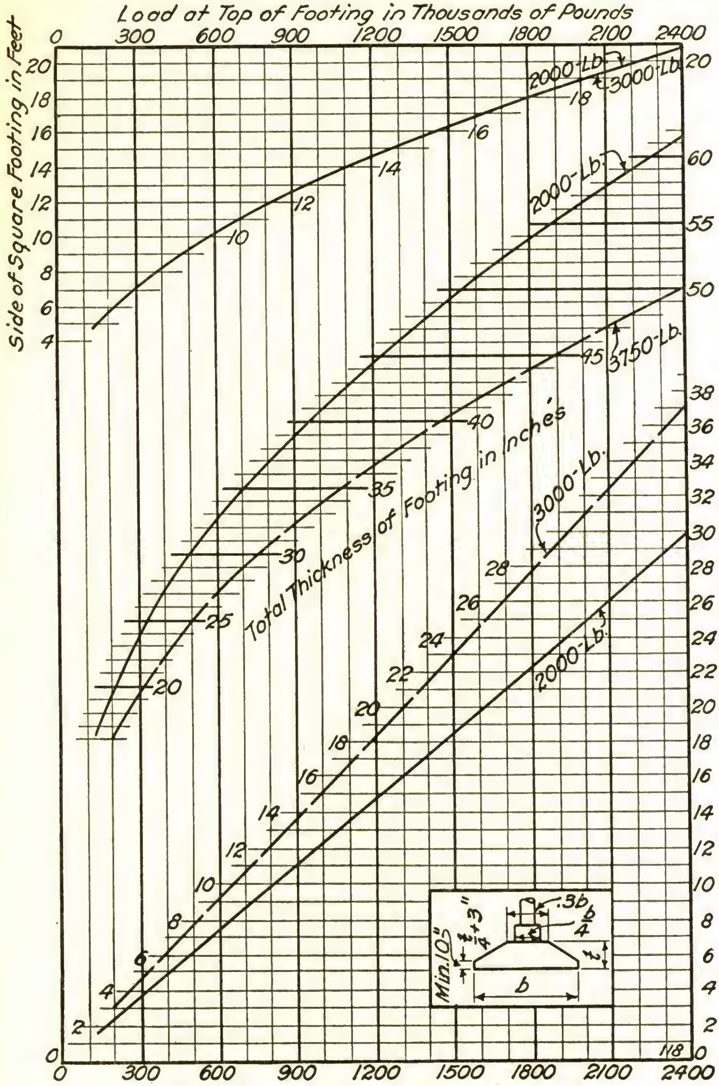
DIAGRAM 117.—5000-LB. SOIL—2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Fig. 16, page 148.

DESIGN OF SLOPED-TOP FOOTINGS WITH $v_c = 0.03 f'_c$

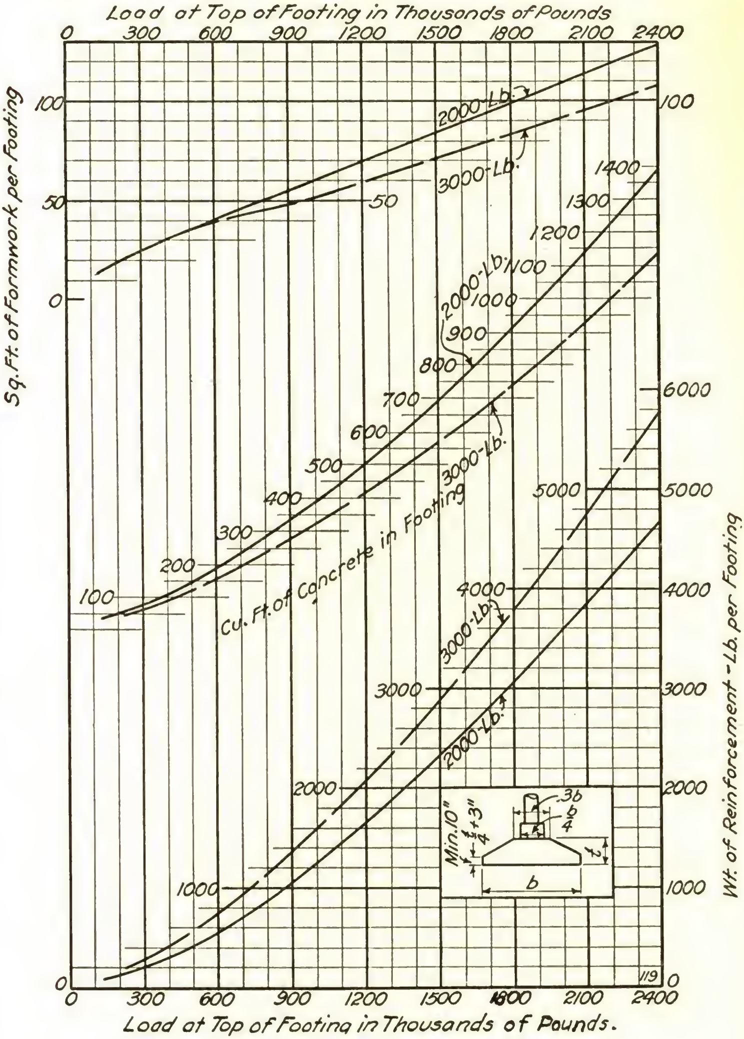
DIAGRAM 118.—6000-LB. SOIL — 2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Diagram 111, page 151.

QUANTITIES FOR SLOPED-TOP FOOTINGS WITH $v_c = 0.03f'_c$

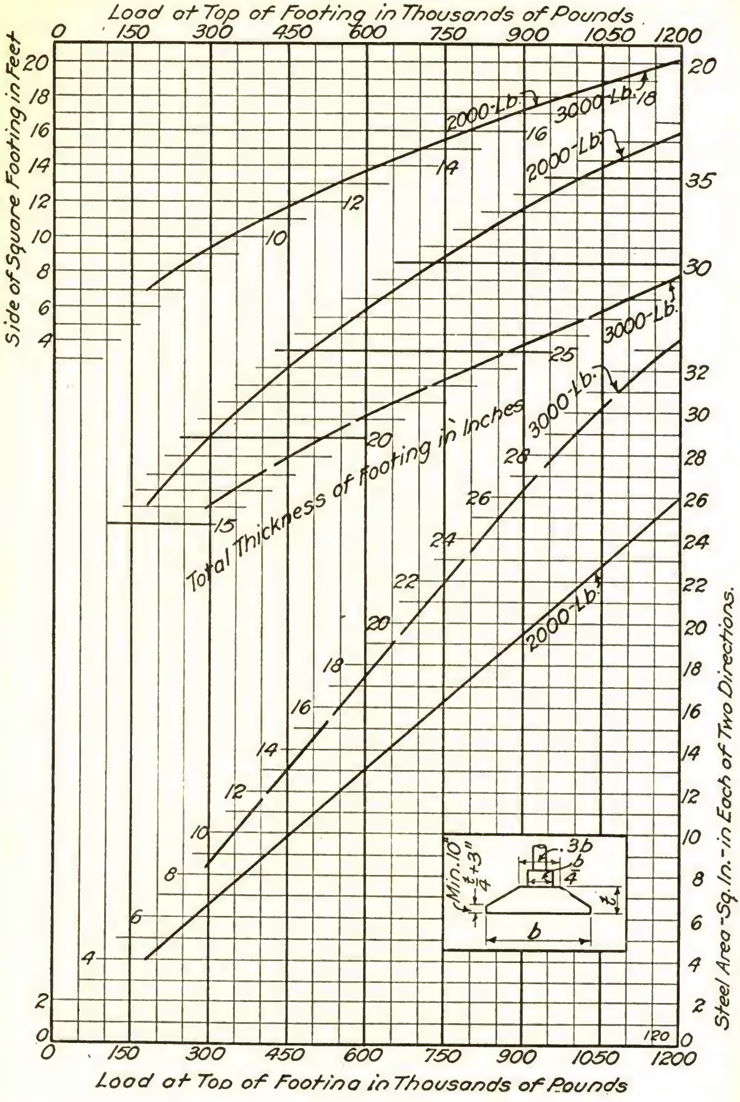
DIAGRAM 119.—6000-LB. SOIL—2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Fig. 16, page 148.

DESIGN OF SLOPED-TOP FOOTINGS WITH $v_c = 0.03f'_c$

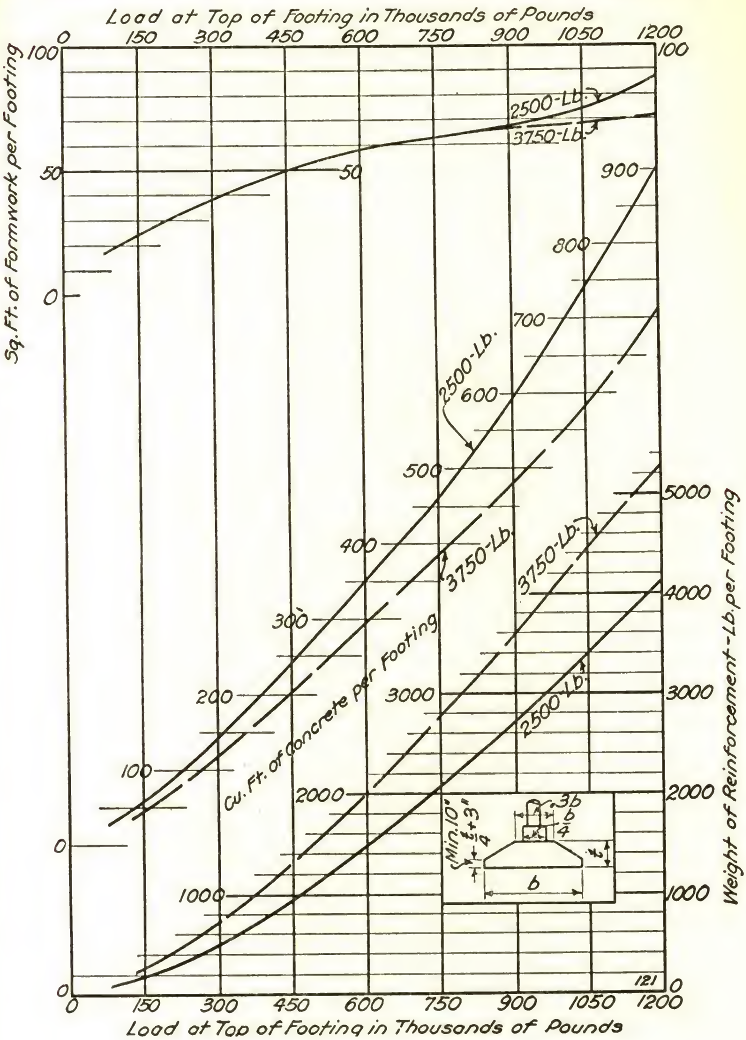
DIAGRAM 120.—3000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Diagram 111, page 151.

QUANTITIES FOR SLOPED-TOP FOOTINGS WITH $v_c = 0.03f'_c$

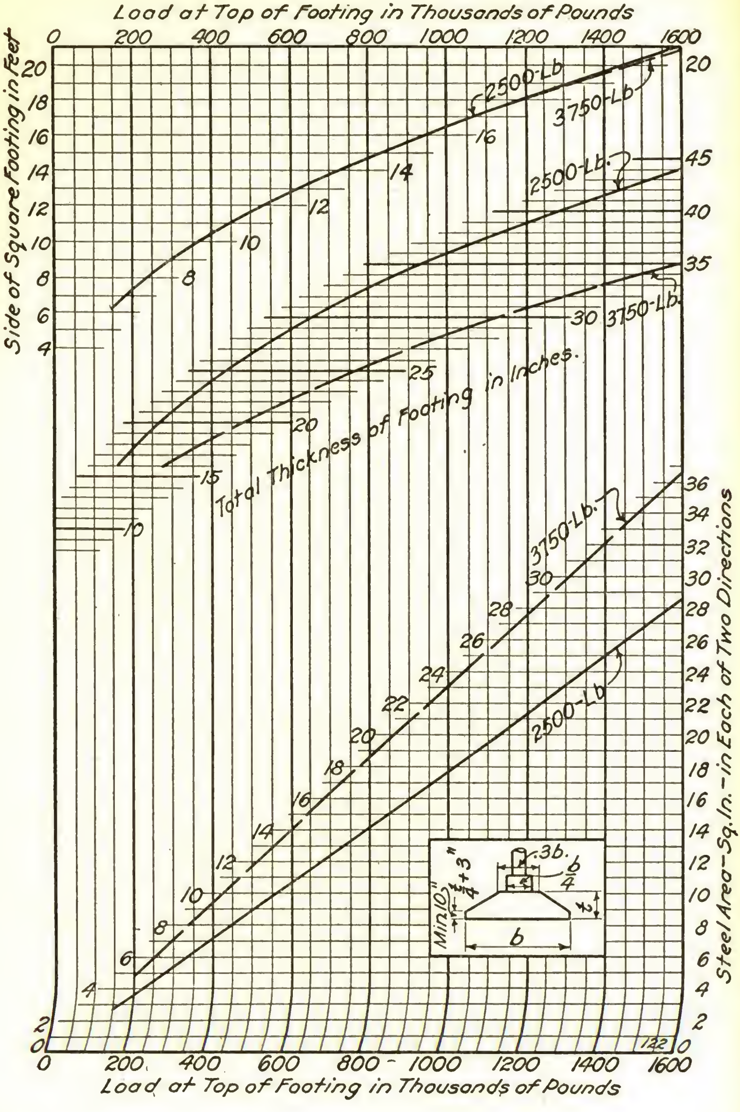
DIAGRAM 121.—3000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Fig. 16, page 148.

DESIGN OF SLOPED-TOP FOOTINGS WITH $v_c = 0.03f'_c$

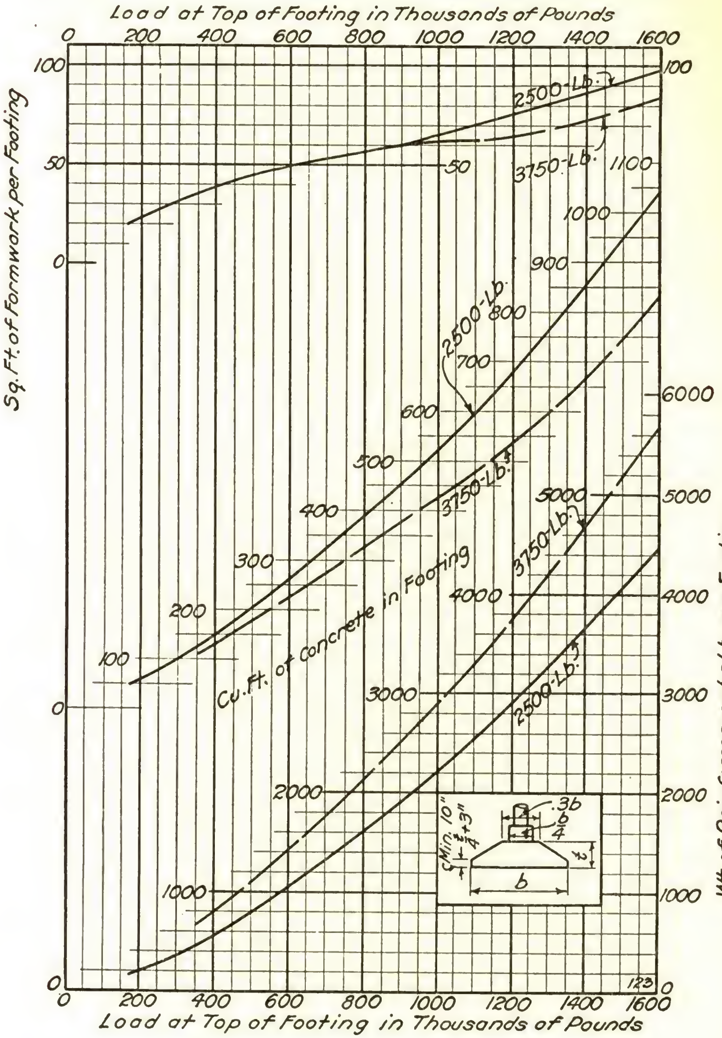
DIAGRAM 122.—4000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Diagram 111, page 151.

QUANTITIES FOR SLOPED-TOP FOOTINGS WITH $v_c = 0.03f'_c$

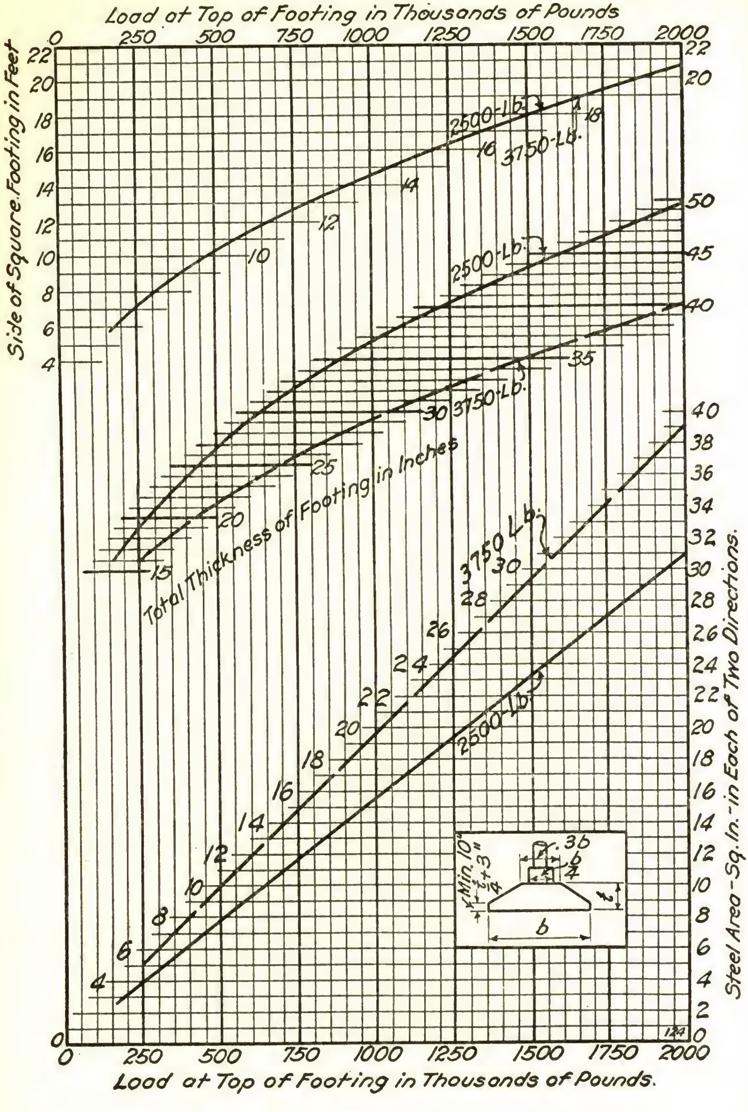
DIAGRAM 123.—4000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Fig. 16, page 148.

DESIGN OF SLOPED-TOP FOOTINGS WITH $v_c = 0.03f'_c$

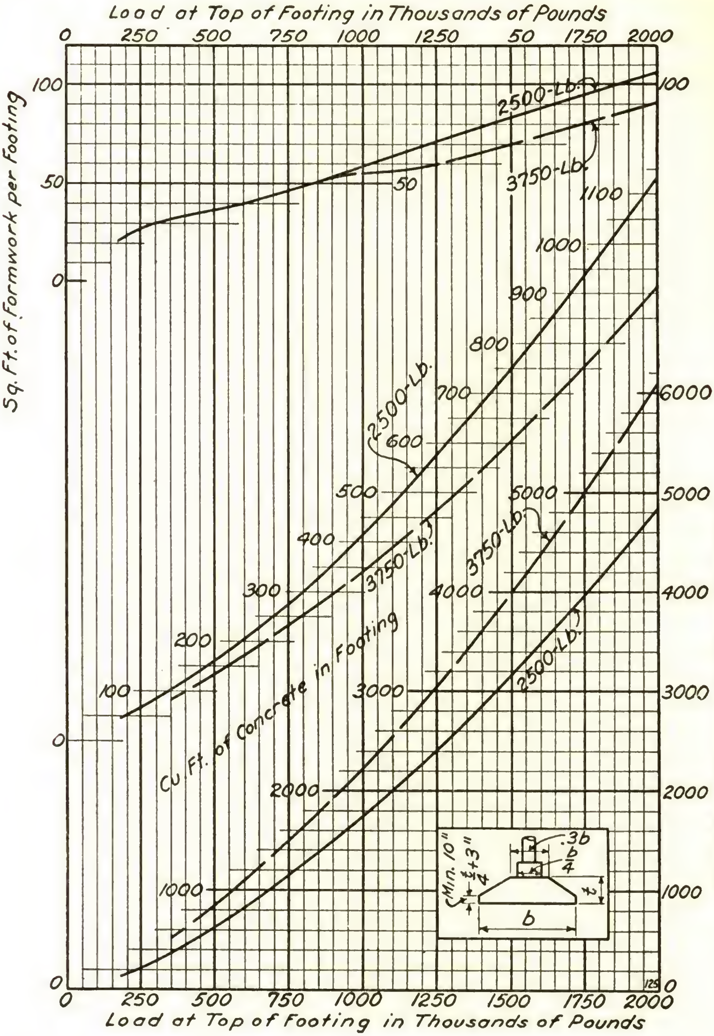
DIAGRAM 124.—5000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Diagram 111, page 151.

QUANTITIES FOR SLOPED-TOP FOOTINGS WITH $v_c = 0.03f'_c$

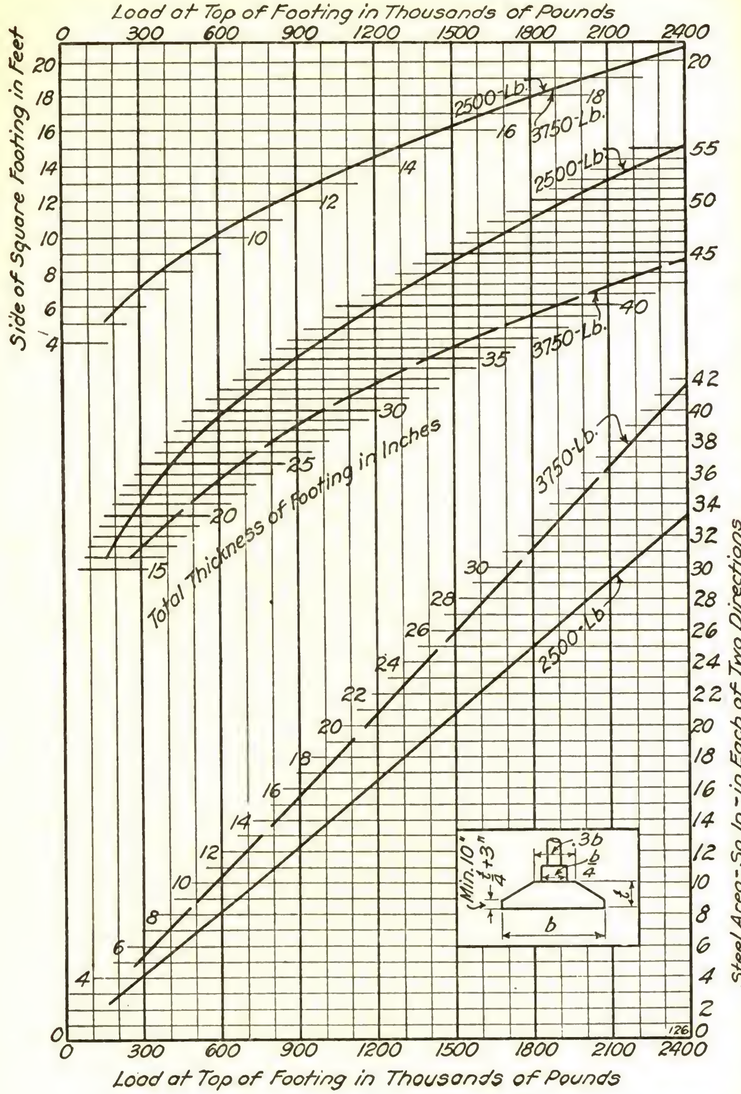
DIAGRAM 125.—5000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Fig. 16, page 148.

DESIGN OF SLOPED-TOP FOOTINGS WITH $v_c = 0.03f'_c$

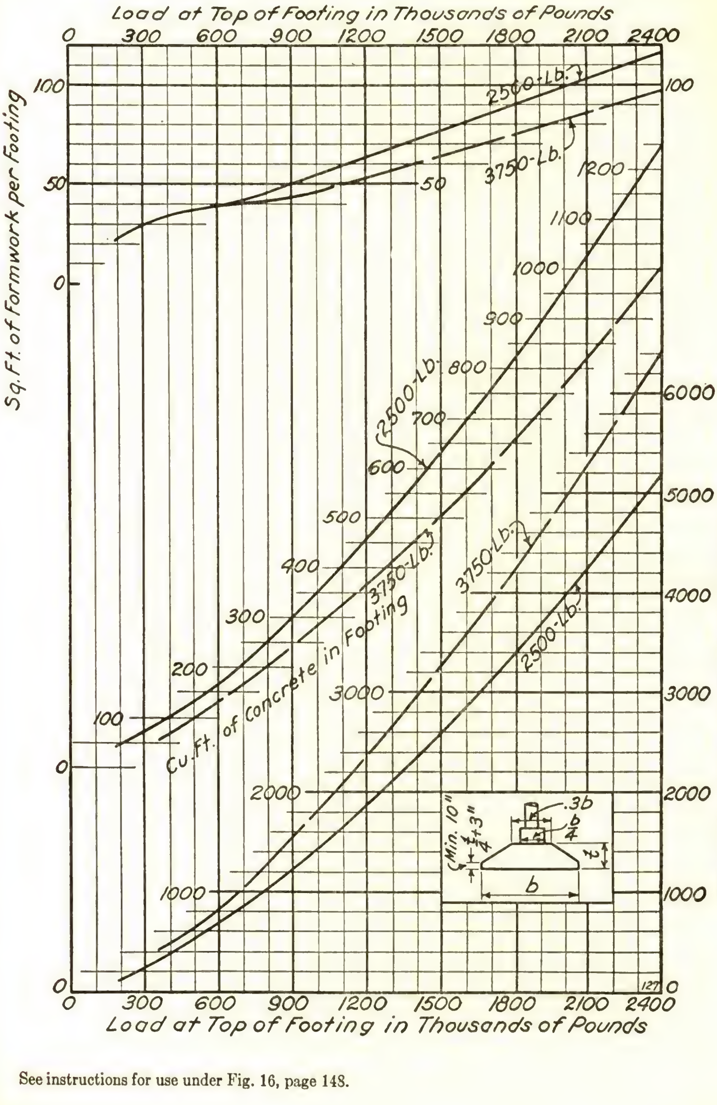
DIAGRAM 126.—6000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Diagram 111, page 151.

QUANTITIES FOR SLOPED-TOP FOOTINGS WITH $v_c = 0.03f'_c$

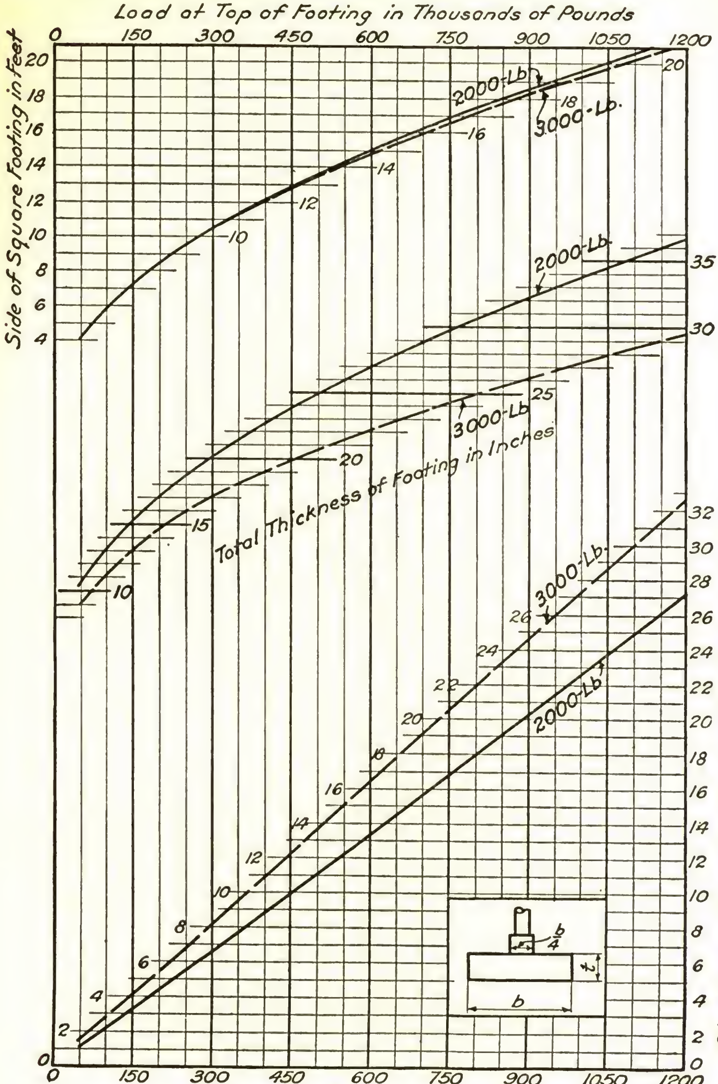
DIAGRAM 127.—6000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Fig. 16, page 148.

DESIGN OF FLAT-TOP FOOTINGS WITH $v_c = 0.03 f'_c$

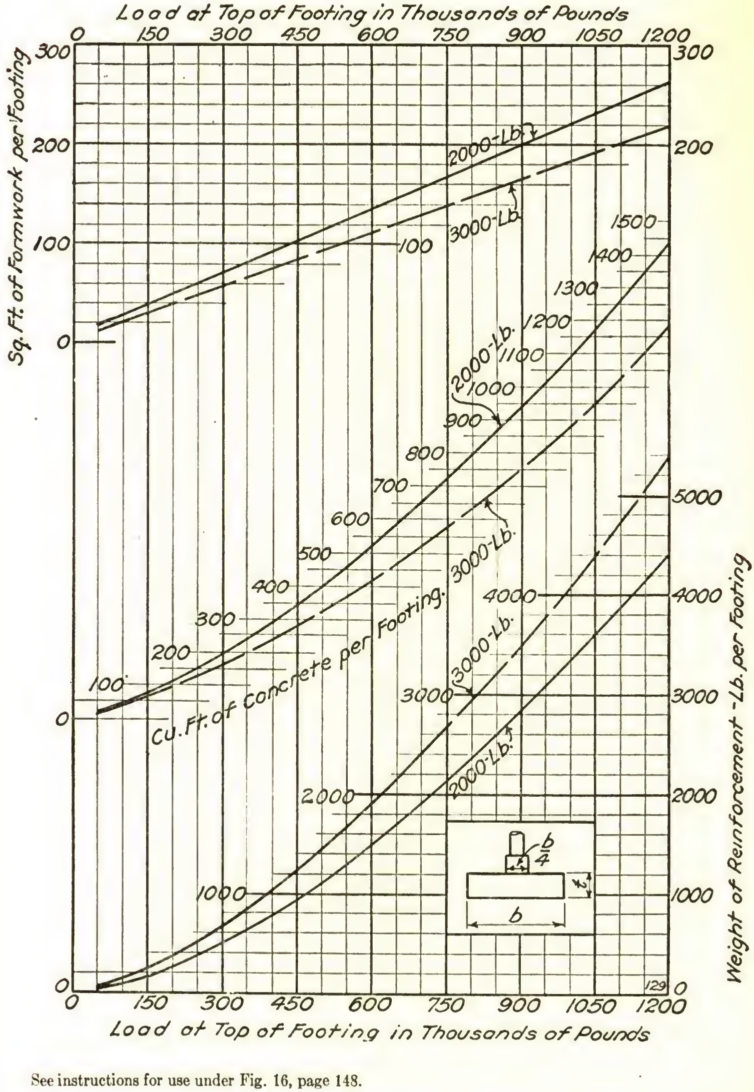
DIAGRAM 128.—3000-LB. SOIL — 2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Diagram 111, page 151.

QUANTITIES FOR FLAT-TOP FOOTINGS WITH $v_c = 0.03f'_c$

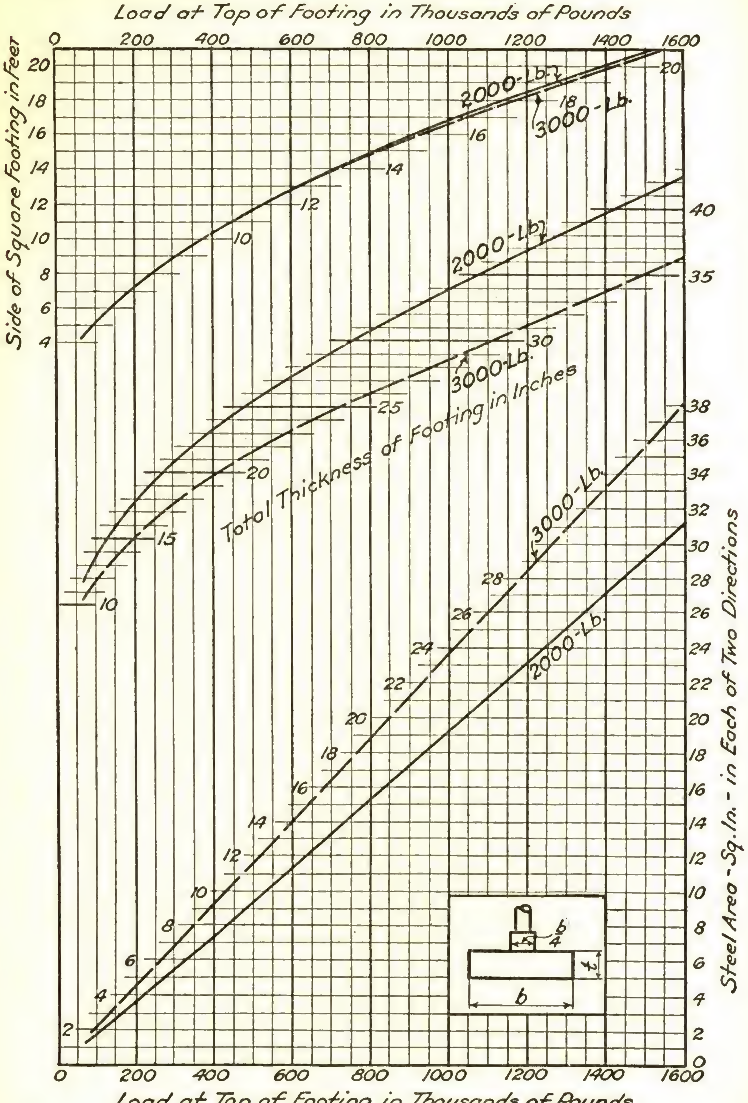
DIAGRAM 129.—3000-LB. SOIL—2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Fig. 16, page 148.

DESIGN OF FLAT-TOP FOOTINGS WITH $v_c = 0.03 f'_c$

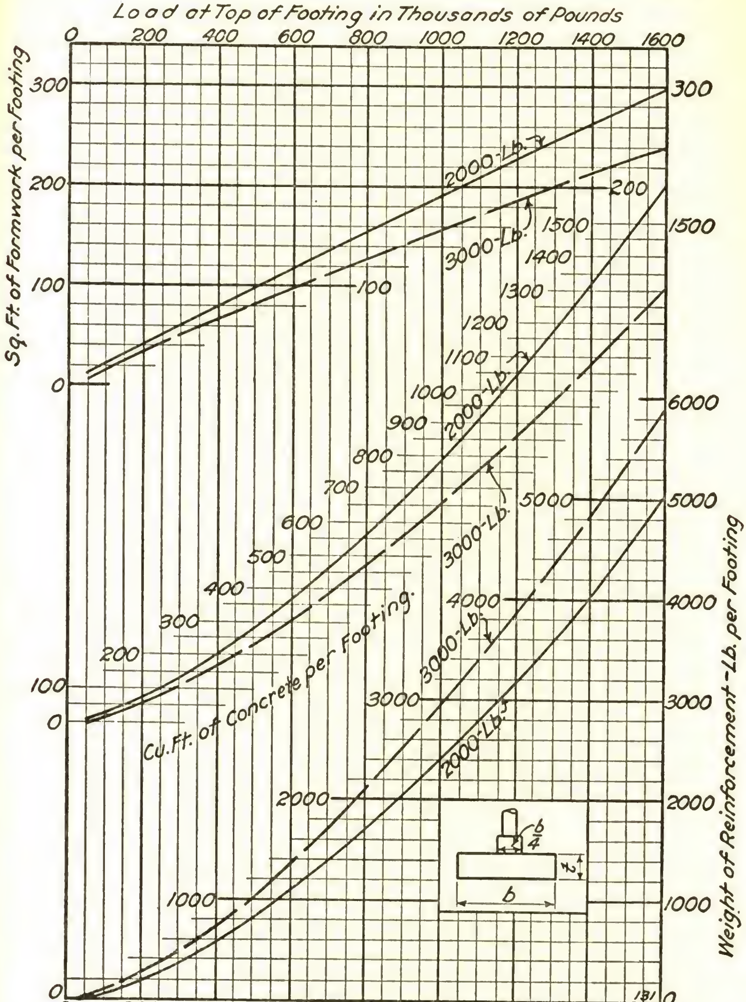
DIAGRAM 130.—4000-LB. SOIL — 2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Diagram 111, page 151.

QUANTITIES FOR FLAT-TOP FOOTINGS WITH $v_c = 0.03f'_c$

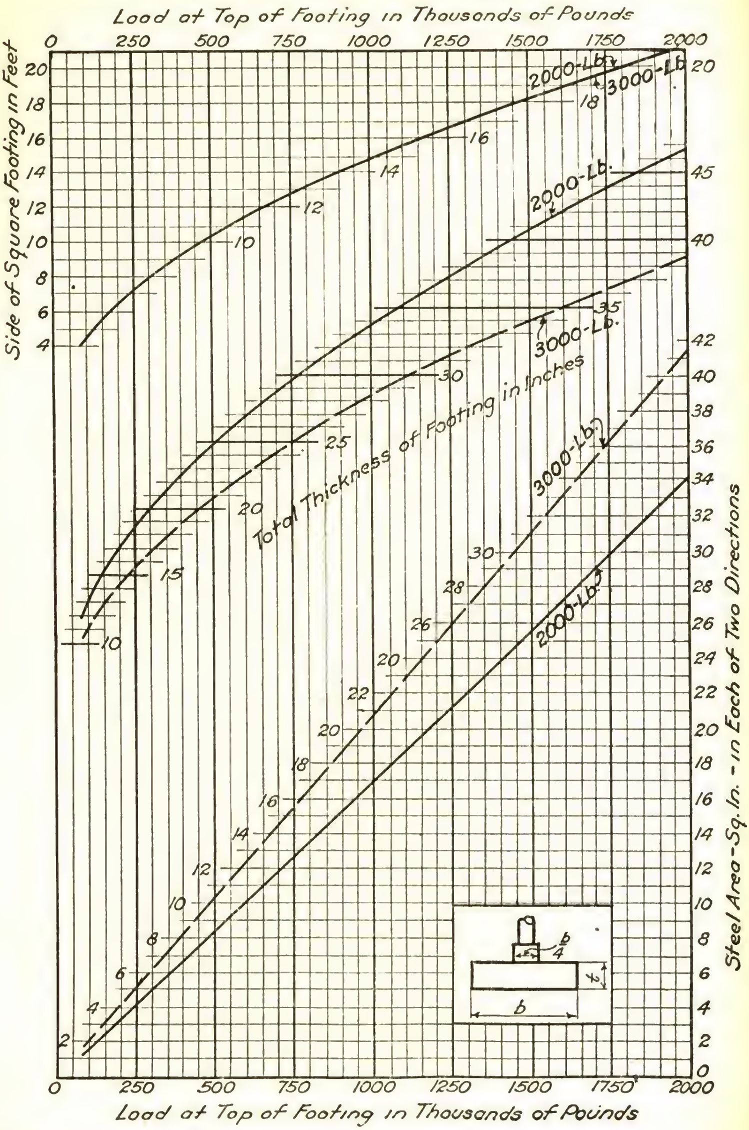
DIAGRAM 131.—4000-LB. SOIL—2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Fig. 16, page 148.

DESIGN OF FLAT-TOP FOOTINGS WITH $v_c = 0.03 f'_c$

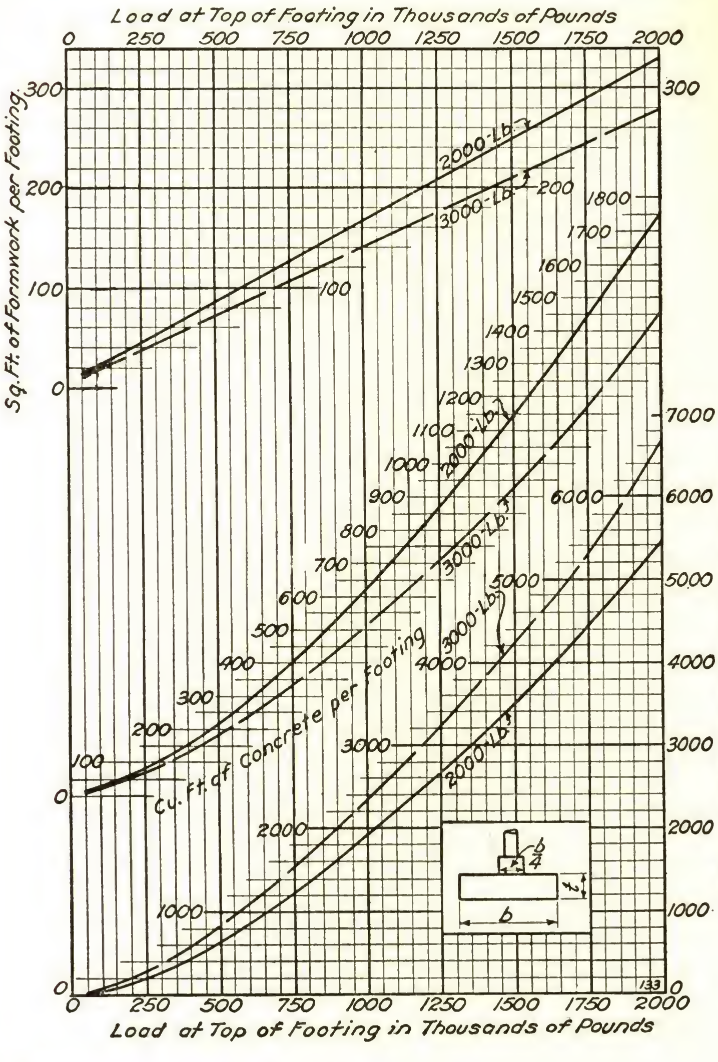
DIAGRAM 132.—5000-LB. SOIL — 2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Diagram 111, page 151.

QUANTITIES FOR FLAT-TOP FOOTINGS WITH $v_c = 0.03f_c'$

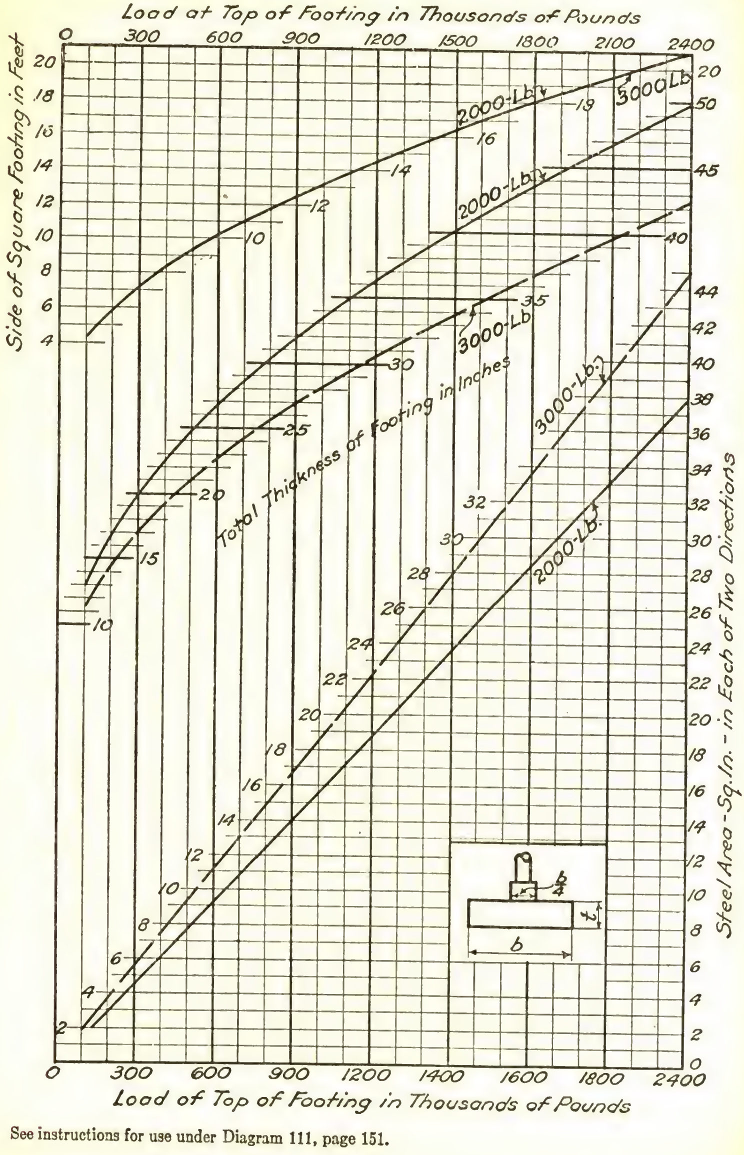
DIAGRAM 133.—5000-LB. SOIL—2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Fig. 16, page 148.

DESIGN OF FLAT-TOP FOOTINGS WITH $v_c = 0.03 f'_c$

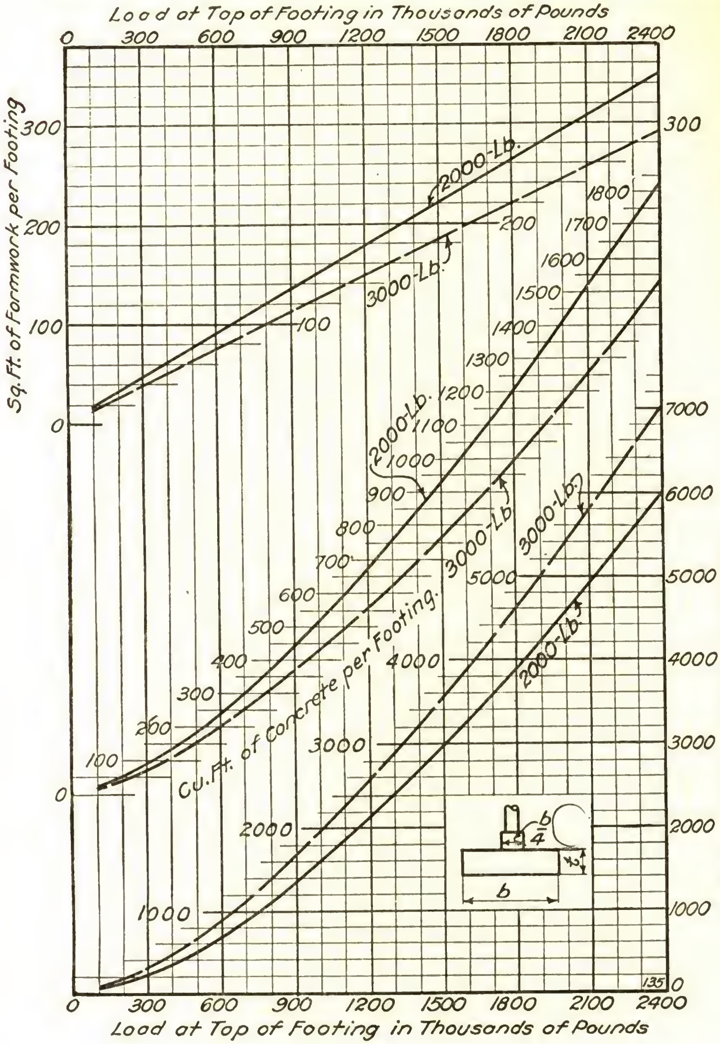
DIAGRAM 134.—6000-LB. SOIL — 2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Diagram 111, page 151.

QUANTITIES FOR FLAT-TOP FOOTINGS WITH $v_c = 0.03f'_c$

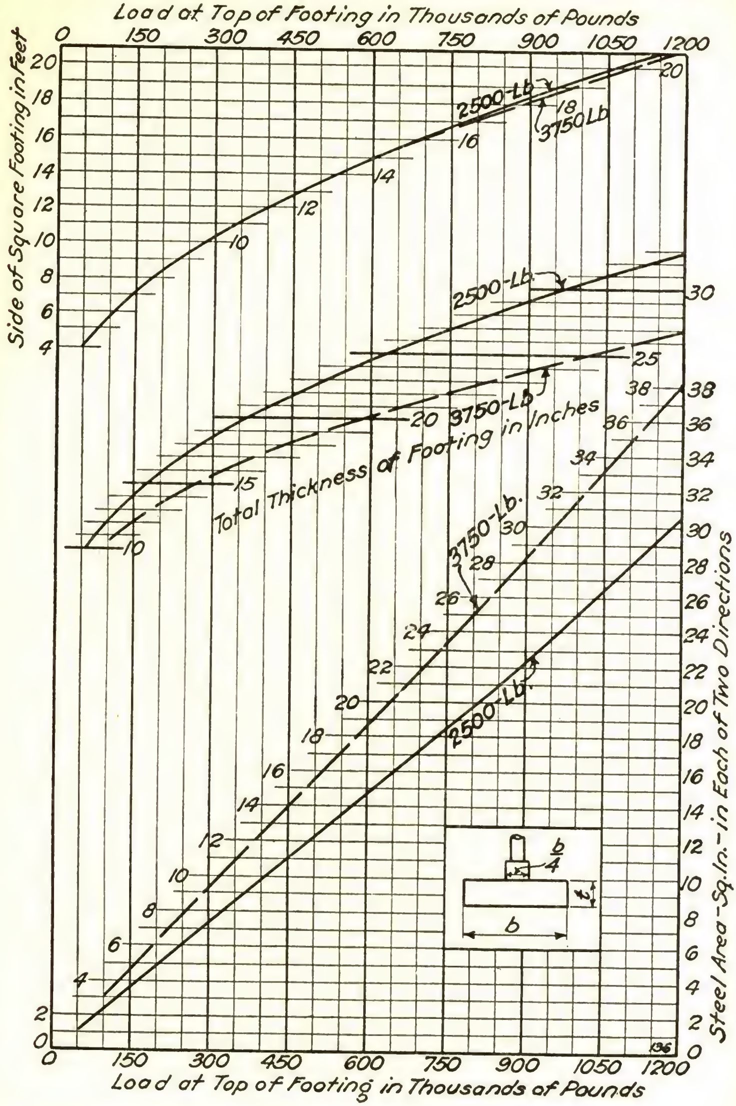
DIAGRAM 135.—6000-LB. SOIL—2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Fig. 16, page 148.

DESIGN OF FLAT-TOP FOOTINGS WITH $v_c = 0.03f'_c$

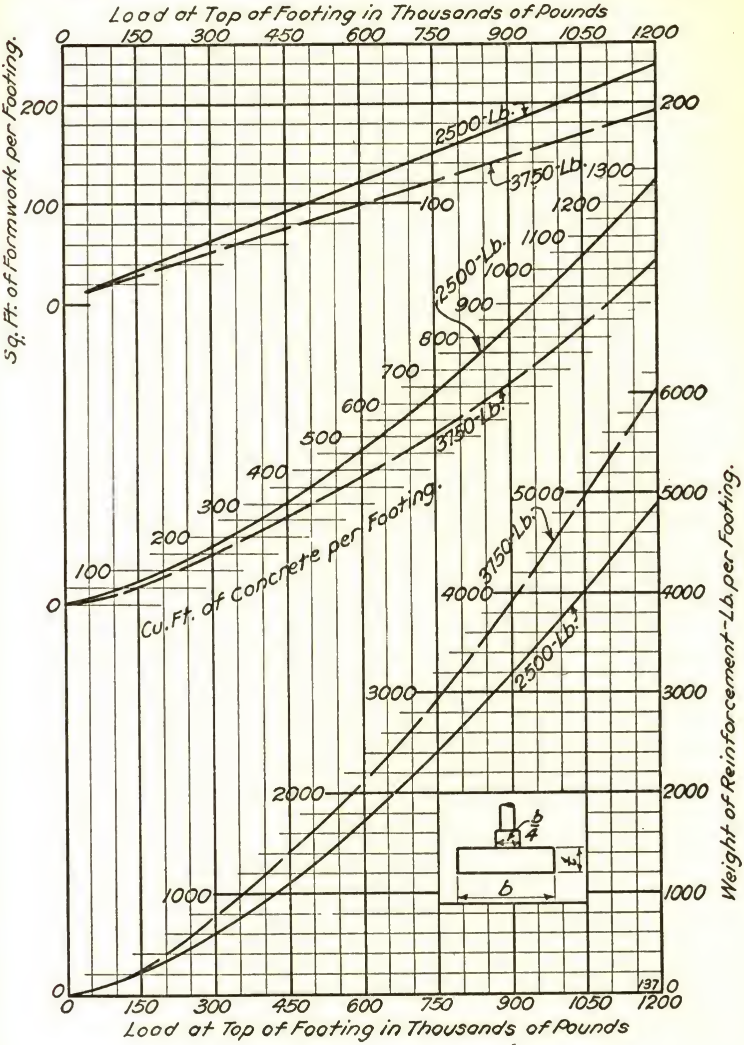
DIAGRAM 136.—3000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Diagram 111, page 151.

QUANTITIES FOR FLAT-TOP FOOTINGS WITH $v_c = 0.03f'_c$

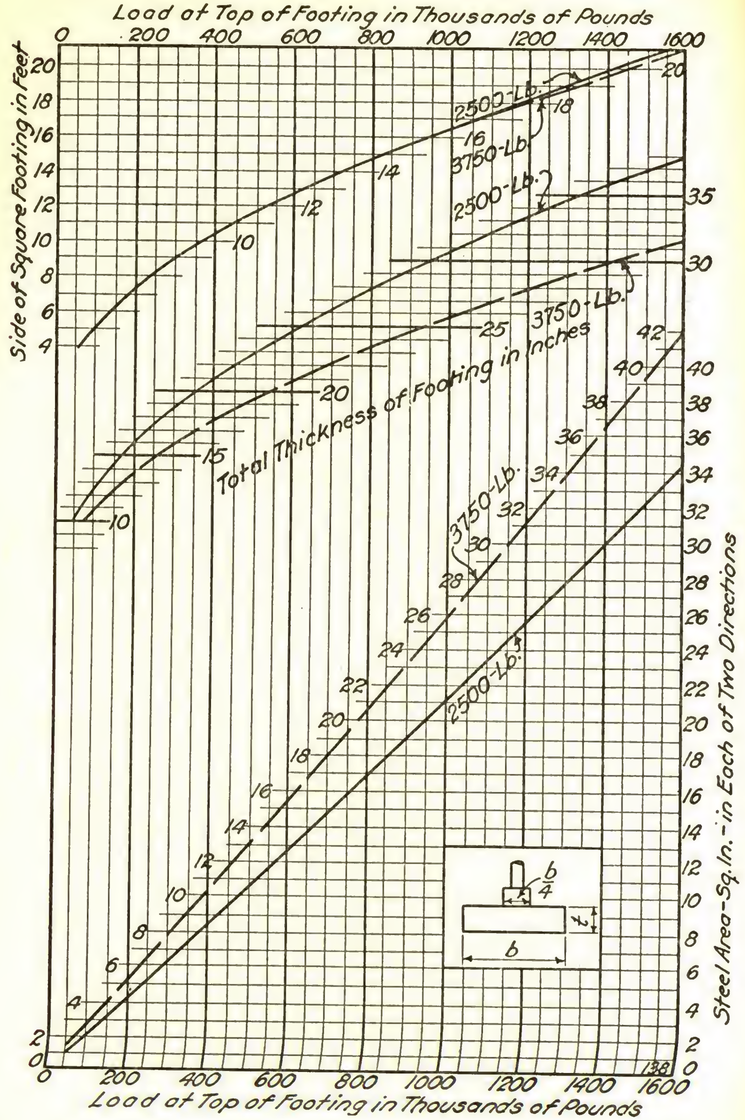
DIAGRAM 137.—3000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Fig. 16, page 148.

DESIGN OF FLAT-TOP FOOTINGS WITH $v_c = 0.03f'_c$

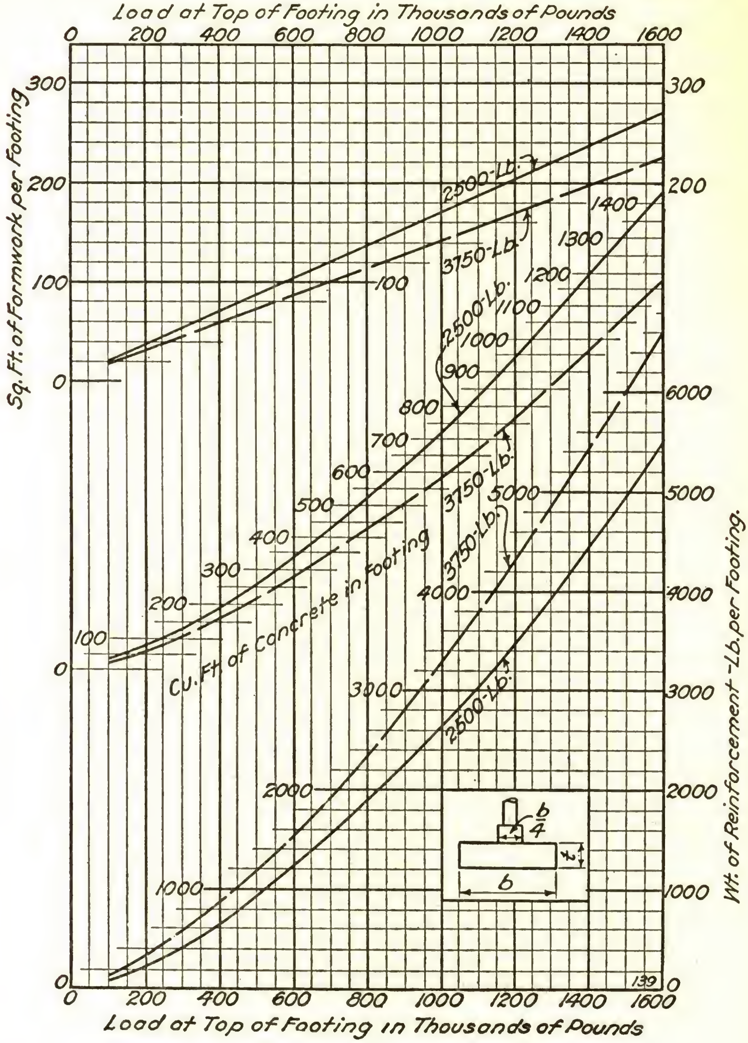
DIAGRAM 138.—4000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Diagram 111, page 151.

Quantities for Flat-Top Footings with $v_c = 0.03f'_c$.

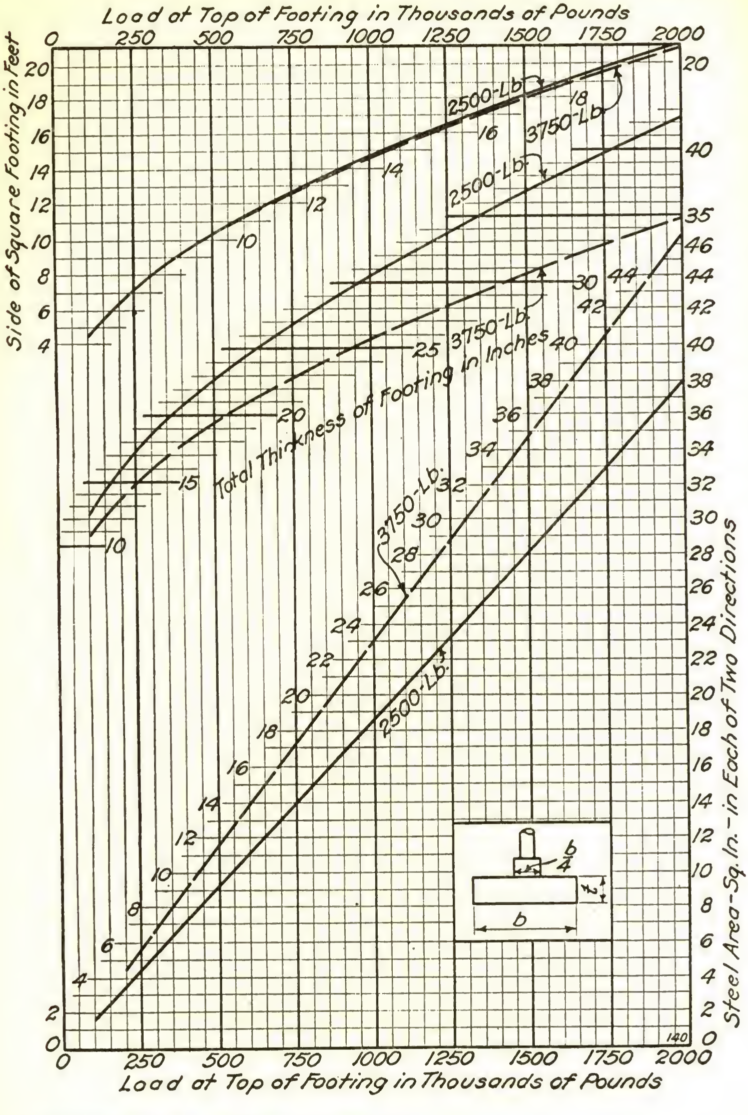
Diagram 139.—4000-lb. Soil—2500-lb. and 3750-lb. Concrete



See instructions for use under Fig. 16, page 148.

DESIGN OF FLAT-TOP FOOTINGS WITH $v_c = 0.03f'_c$

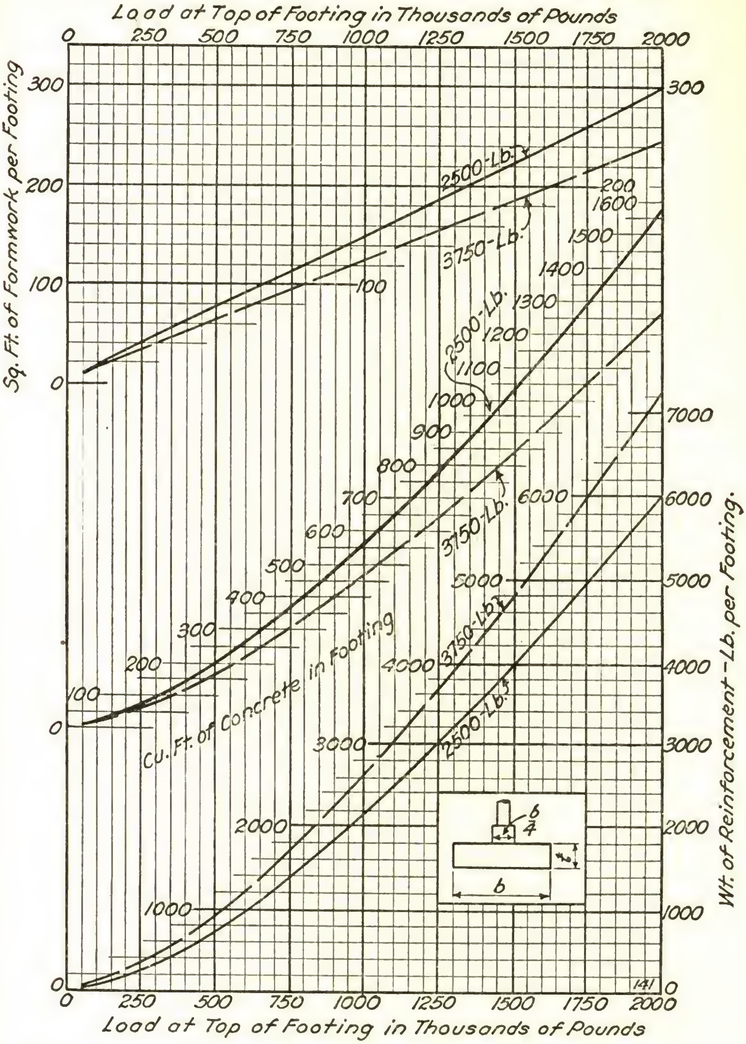
DIAGRAM 140—5000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Diagram 111, page 151.

QUANTITIES FOR FLAT-TOP FOOTINGS WITH $v_c = 0.03f'_c$

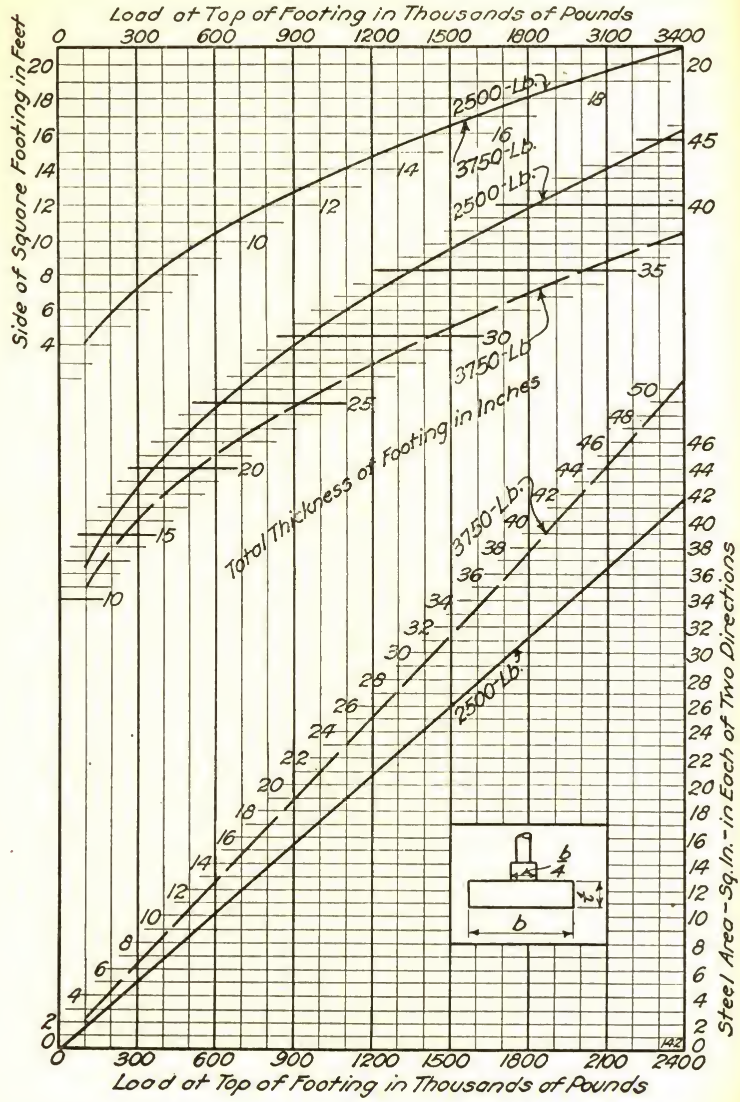
DIAGRAM 141.—5000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Fig. 16, page 148.

DESIGN OF FLAT-TOP FOOTINGS WITH $v_c = 0.03f'_c$

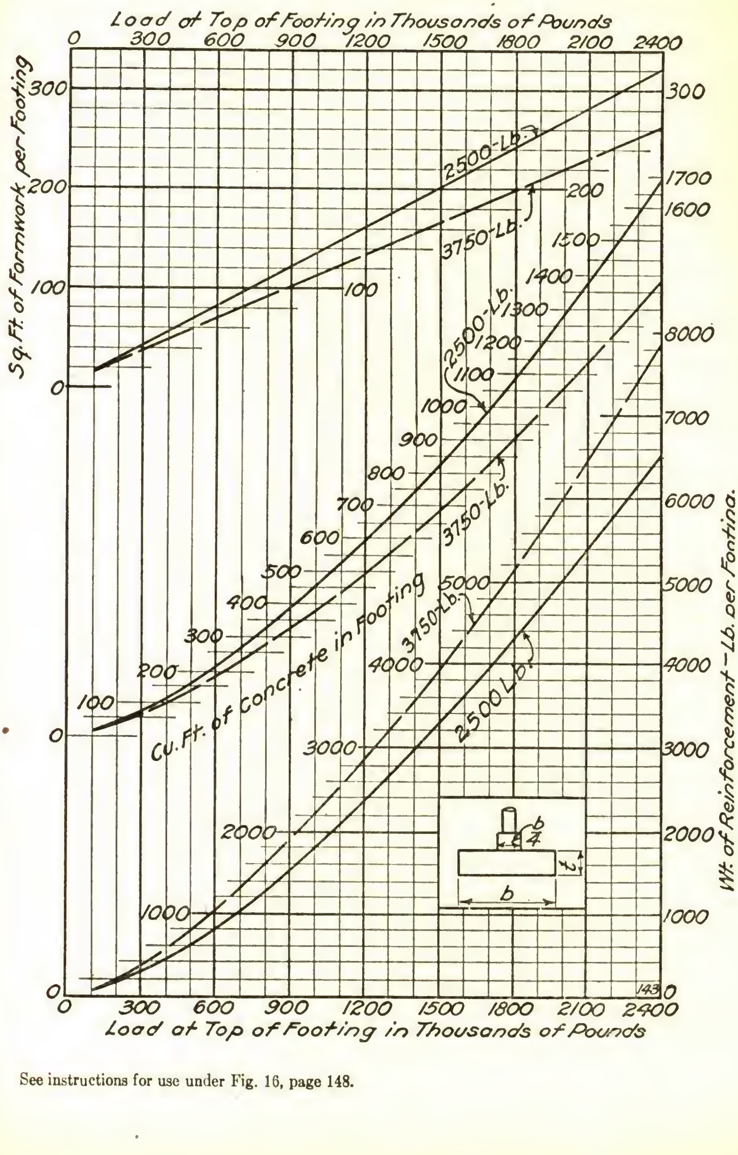
DIAGRAM 142.—6000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Diagram 111, page 151.

QUANTITIES FOR FLAT-TOP FOOTINGS WITH $v_c = 0.03f'_c$

DIAGRAM 143.—6000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Fig. 16, page 148.

PART TWO—COST DATA.

The cost of reinforced-concrete building construction designed in accordance with the 1928 Joint Standard Building Code will vary between fairly wide limits in different parts of the country. Where some cities pay $2.50 or more per cubic yard for concrete aggregate it may sometimes be secured on country jobs for as low as 60 cents. The price of cement varies considerably and the labor cost is subject to great variations. Many other factors, including plant, size of job, number of stories, contracting ability, weather and working conditions, and labor rules contribute to the variation in building cost. I have therefore given in Tables 145 to 156 the *quantities* of concrete, reinforcing steel, formwork, excavation, etc., from which the engineer may compute for himself with little trouble the local price differential for the conditions on which he must operate. In Table 144 and Fig. 18 unit prices have been used, as given later, and the cost comparison given in terms of money.

I have made complete designs of a typical interior panel, from foundation to roof, for 96 buildings covering the following range:

2 Types of Construction	:	Flat Slab and Beam-and-Girder.
2 Concrete Strengths	:	2,000 lb. and 3,000 lb.
2 Panel Sizes	:	18 ft. by 18 ft. and 22 ft. 6 in. by 22 ft. 6 in.
2 Live Loads	:	100 lb. and 300 lb. per sq. ft.
2 Soil Pressures	:	3,000 lb. and 6,000 lb. per sq. ft.
3 Building Heights	:	6, 9 and 12 stories.

In selecting the values for the variables, two choices have been made representing a fairly low and a fairly high value. Thus an 18-ft. square panel is fairly small while a 22-ft. 6-in. square panel is fairly large. A 100-lb. live load is small, a 300-lb. live load is large. A six-story building is quite low and a twelve-story building fairly tall, as current work goes.

The quantities reported are for the structural framework of the building, including roof and floor slabs, but not including exterior walls, brick masonry, architectural finish or equipment of any kind. The design was carried out in general conformity with the Joint Code, using the tables and diagrams given in this paper. In designing, the size of column has been maintained the same for all parallel designs between the usual 2,000-lb. concrete structure and the structure using stronger concrete. The usual design, however, involves 3,000-lb. concrete in the lower story columns while the higher-concrete-strength design uses 2,000-lb. concrete in the top stories increasing to 5,000-lb. concrete in the lower stories. The roof, floor slabs and the footings are designed for one concrete strength in any one building—2,000 lb. or 3,000 lb. as the tables indicate. It would be possible to manipulate such a comparison to a considerable extent, but my intention, carried out with much care and thought, has been to make designs in the 2,000-lb. and 3,000-lb. classes, which would represent equally good practice in both cases and which would show only that saving in materials which naturally results from the use of richer concrete. One of the chief savings is the reduction in dead weight of the structure itself which shows up mainly in the column and footing quantities. The saving resulting from the use of richer concrete may be realized in either one of two ways: (1) as in done in this paper, the first cost of the building may be reduced, or (2) the size of columns may be greatly reduced resulting in a valuable increase in the useable floor areas.

In Tables 145 to 152 inclusive, quantities are stated for the elements of which buildings are composed—roof, floors, columns, footings—and numerous buildings may be worked out from these tables which are not included in Tables 153 to 156 inclusive, where the total quantities for one panel from footing to roof for 96 buildings have been assembled. The concrete quan-

tities do not include a basement floor on the ground. Formwork has been given in square feet of concrete surface in contact with the forms, except that steel column forms are given in units for one story. Reinforcing steel is separated, between bars (both straight and bent) and spiral (including ties and stirrups in the spiral item). Excavation allows 4 inches on all sides of the footing and excludes 6 inches of the combined depth of footing and pedestal. This is pit excavation only. In four cases where 3,000-lb. soil would not support the column load within the panel area, concrete piles are indicated, computed for a load of 30 tons each. Items containing piles suffer a considerable increase in cost, but in all cases where piles have been required with 2,000-lb. concrete they have been used also with the 3,000-lb. concrete, so that the difference in price is not greatly affected. In actual designing, cases may arise where the saving in dead weight will make it possible to use spread footings with 3,000-lb. concrete as against piles for 2,000-lb. concrete and so make large savings not shown in this comparison. With this information the proper unit cost for each item may be readily determined by an experienced estimator.

The unit costs which I have used in Table 144 and Fig. 18 are as follows:

Reinforcing bars in place \$70.00 per ton.
Spiral, ties and stirrups in place \$90.00 per ton.
Wood formwork, erected and removed, 25¢ per sq. ft.
Steel column forms, rental and labor, \$15.00 per column per story.
Hand excavation, including backfill, \$2.70 per cu. yd.
Concrete piles, 30 tons capacity, \$42.00 each.

2,000-lb. concrete in place .... \$10.95 per cu. yd.

1.35 bbl. cement and sacks .... at \$2.75 = \$3.72
0.59 cu. yd. fine aggregate .... at 2.50 = 1.48
0.70 cu. yd. coarse aggregate .... at 2.50 = 1.75
Water (7½ gal. per sack)
Mixing and placing, including plant and other overhead charges and cost of protecting and curing .... at 4.00

3,000-lb. concrete in place .... \$11.66 per cu. yd.

1.65 bbl. cement and sacks .... at \$2.75 = \$4.54
0.56 cu. yd. fine aggregate .... at 2.50 = 1.40
0.69 cu. yd. coarse aggregate .... at 2.50 = 1.72
Water (6 gal. per sack)
Mixing and placing, including plant and other overhead charges and cost of protecting and curing .... at 4.00

5,000-lb. concrete in place .... \$13.47 per cu. yd.

2.4 bbl. cement and sacks .... at \$2.75 = \$6.60
0.41 cu. yd. fine aggregate .... at 2.50 = 1.02
0.74 cu. yd. coarse aggregate .... at 2.50 = 1.85
Water (4½ gal. per sack)
Mixing and placing, including plant and other overhead charges and cost of protecting and curing .... at 4.00

The quantities in the mixes given above are based on tests in which both gravel and crushed limestone were used for coarse aggregates and

in which the slumps varied from 5 in. to 7 in. The maximum size of the aggregate was ¾ in.

Advantages of Stronger Concrete.—Engineers are so accustomed to thinking of concrete for buildings in terms of 2,000-lb. strength at 28 days that it may be novel to consider using a 3,000-lb. concrete as the basic mix. For a great many years, however, it has been general practice to use 3,000-lb. concrete in the more heavily loaded columns of buildings. With the continually widening use of scientific concrete in recent years, many engineers and contractors have discovered that the full economy of scientific concreting can be secured in 2,000-lb. strength only at the sacrifice of workability. In order to keep a concrete of easy working quality on the job, contractors are generally—under usual weather conditions—providing a concrete of about 2,500-lb. strength in structures where only 2,000-lb. concrete is required. This experience has led many engineers to design structures in 2,500 or 3,000-lb. concrete in order to utilize the strength actually provided on the job. My own experience has led me to believe that a slight addition in cement (and a big reduction in water) as compared with the old-time practice, will produce a 3,000-lb. concrete which is more workable and less permeable at a very small increase in cost over the 2,000-lb. variety Even if such a concrete should cost as much in terms of the completed building as 2,000-lb. concrete, it would be preferable for several reasons. The increased workability due to added cement (as compared with 2,000-lb. concrete) will facilitate placing and surfacing and so overcome the objection of the field forces to stiff consistencies. The added cement and decreased water will greatly decrease the porosity and permeability and increase the weather resistance of the concrete. Such concrete will attain strength more rapidly and as a result will permit an earlier finishing of the work and require protection against low temperatures for a shorter period. It will also afford a much better bond with the reinforcing steel, and thus permit the use of larger bar sizes and shorter bars. It will decrease the thickness of members as controlled by compression and diagonal tension and decrease the weight of the supporting structure. The shrinkage, while greater than that for 2,000-lb. concrete, will be considerably less than that of the old-time wet 1 : 2 : 4 concrete. In the case of 5,000-lb. concrete as used in heavily loaded columns, the actual amount of cement and the shrinkage will be less than with the old-time wet 1 : 1 : 2 (3,000-lb.) concrete. This shrinkage consideration will limit the use of *exceedingly* rich mixes, especially in slabs and walls of great length, but it will not deter from the use of properly made 3,000-lb. concrete in floors and walls or of properly made 5,000-lb. concrete in columns, since it involves less shrinkage than existing buildings are carrying comfortably.

Flat Slab Cost Study.—A study of tables Nos. 145 and 146 or of 147 and 148 shows that there is but a small difference in cost as between a four-way flat slab floor (or roof) made of 2,000-lb. or 3,000-lb. concrete. While these quantities are based on actual computations by the four-way system, the provisions of the Code are such that no material differences would be expected from computations by the two-way system. At the unit prices given before the saving with stronger concrete is from 1.0 to 1.8 cents per sq. ft. of floor area. The real saving is secured in the columns and footings through the reduction in dead weight. The quantities in tables Nos. 153 and 154 are priced and plotted in Fig. 18. A study of the data indicates in a general way the following comparisons for complete buildings:

(1) A considerable saving in cost for all loads and spans may be effected by the use of 3,000-lb. concrete.

(2) This saving increases (but at a decreasing rate of increase) with the number of stories in the building.

(3) This saving increases as the soil pressure decreases.

(4) This saving increases slightly with larger panel sizes.
(5) This saving increases markedly with heavier live loads.

The least saving of the 3,000-lb. over the 2,000-lb. concrete (smaller panel, lighter live load, higher soil pressure) amounts to 1.8 cents per sq. ft. of total floor area for a six-story building and increases to 3.3 cents per sq. ft. for a twelve-story building. The greatest saving (larger panel,

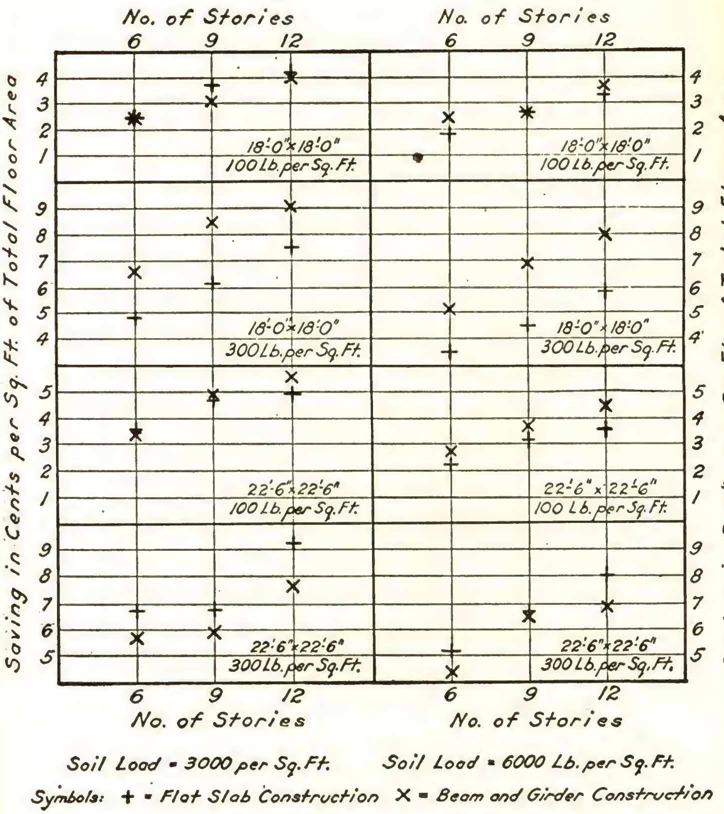


FIG. 18.—SAVING FROM USE OF 3,000-LB. CONCRETE.

heavier live load, lower soil pressure and higher building) amounts to 9.2 cents per sq. ft. for a twelve-story building. The maximum saving, expressed in percentage of the cost of the concrete frame is over 7 per cent. The least saving is over 2 per cent.

Beam-and-Girder Cost Study.—In this study of the "beam-and-girder" system the framing of the members was as follows: Girders from column to column in one direction, beams directly from column to column in the other direction, two beams (at the third points) between these marginal beams and one-way slabs spanning between all beams. The slabs have temperature and shrinkage reinforcement as required by the Joint Code. A

study of Tables 149 and 150, or of 151 and 152 shows that there is an appreciable saving in cost in the floor and roof construction itself by the use of the 3,000-lb. concrete. At the unit prices given before this saving amounts to from 1.6 to 3.5 cents per square foot. There is also a reduction in the dead weight of the construction, resulting in further savings in the cost of colums and foundations. The quantities of Tables 155 and 156 are priced and plotted in Fig. 18. A study of these data indicates in a general way for complete buildings:

(1) A considerable saving for all loads and spans may be effected by the use of 3,000-lb. concrete.

(2) This saving increases (but at a decreasing rate) with the number of stories in the building.

(3) This saving increases as the soil pressure decreases.

(4) This saving increases as the panel size increases with 100-lb. live-load, but decreases as the panel size increases with 300-lb. live-load.

(5) This saving increases markedly with heavier live loads.

The least saving (smaller panel, lighter live load, higher soil pressure) amounts to 2.5 cents per sq. ft. of total floor area for a six-story building and increases to 4.0 cents per sq. ft. for a twelve-story building. The greatest saving (smaller panel, heavier live load, lower soil pressure and higher building) amounts to 9.1 cents per sq. ft. for a twelve-story building. Expressed in percentage of cost of the concrete frame, the maximum saving is 7.2 per cent. The least saving is 3.1 per cent.

Comparison of Flat Slab and Beam-and-Girder Construction.—In general the savings effected by the use of 3,000-lb. concrete are greater with beam-and-girder construction than with flat slab. An exception occurs in the case of the larger panel with the heavier live load. The total cost of the beam-and-girder construction is always greater than that of the corresponding flat slab. The flat slab type, as here designed, is the standard form with enlarged column capitals and drop panels at the column head. Where the capital or the dropped panel, or both, must be omitted for architectural reasons the cost of the flat slab type would be greatly increased.

A study of the data of Table 144 leads to the following general conclusions as to the excess in cost of beam-and-girder over standard flat slab construction:

(1) It becomes greater as the load increases.

(2) It becomes greater as the soil pressure increases.

(3) It becomes less as the panel size increases.

(4) It is not greatly affected by the number of stories.

Expressed in figures the beam-and-girder type costs about 9.5 per cent more for the 18-ft. panel, 100-lb. *LL* and 3,000-lb. soil. This difference increases to 16 per cent for the 18-ft. panel, 300-lb. *LL* and 6,000-lb. soil. With 22-ft. 6-in. panel the first figure is reduced to about 3.5 per cent and the second to about 10 per cent.

Comparison With Chicago Code.—I have in my office complete quantities for designs in accordance with the present Chicago Code (now in process of revision) which afford comparisons between the cost of the joint requirements and those of a well-known and widely-used present standard. The great length of this paper forbids anything beyond a very brief statement of this comparison.

For the four-way flat slab type the cost is nearly identical on the basis of 2,000-lb. concrete. The Joint Code, however, will give a considerably better balanced design than the Chicago ordinance. The increased stresses in Joint Code are offset by added anchorage of reinforcement and other safeguards.

For the beam-and-girder type the cost in accordance with the Joint

Code is about 8 per cent less than the Chicago figure. This saving is for a typical interior panel and the saving for the entire building including exterior panels would be somewhat less. The Joint Code requires far better anchorage of reinforcement than the Chicago Code but the clear span moment calculations and the increased stresses more than offset this addition.

Most designers, I feel, will agree that the effect secured by the Joint Code in decreasing the difference in cost between the beam-and-girder and the flat-slab construction is desirable. The present advantage enjoyed by the flat-slab type will still remain but will be decreased in amount by the adoption of the Joint Code.

In the Chicago beam-and-girder designs no shrinkage and temperature reinforcement was included.

Savings from Scientific Concrete Proportioning.—All of the figures in this paper are based on the use of moderately stiff, non-segregating concrete. Such concrete may be manufactured and placed in the field to give any required strength at a considerably less cost than the wet consistencies often used in the past. This added saving acts to increase the final saving that may be realized by the use of 3,000-lb. scientific concrete to values greater than shown in this paper if comparison is made with the cost of old-time sloppy concrete of the same strength.

Use of Cost Data for Estimating.—The data of Tables 145 to 156 inclusive may be useful to engineers in estimating in advance the cost of proposed buildings to be designed under the 1928 Joint Code. In making use of this material for this purpose it is advisable to comply with the following necessary limitations:

(1) Determine proper cost units for the conditions of the contemplated work. The unit costs as given in this paper and on which Table 144 and Fig. 18 are based are not applicable, except by coincidence, to such use. They are too large for large jobs and too small for small jobs. The size of building, number of re-uses of forms, cost of materials and labor, weather conditions, and many local conditions will determine the proper values of cost units in any case.

(2) Proper allowance must be made for exterior and special panels, since these tables deal with typical interior panels exclusively.

(3) Only the pit excavation for footings is included under the item of excavation in the table. The general (steam shovel) excavation must be added, with allowance for overrun at sides of lot.

(4) A small allowance should be made for necessary wastage of material in the final design. In these tables fractional bars have been used occasionally in order to permit of accurate interpolation between the computed values. In a few cases the soil pressure must slightly exceed that given in the tables in order to keep the footings within the panel area.

(5) Column live loads on all floors have been reduced in accordance with Chicago practice—15 per cent for the top floor, 20 per cent for the next, increasing by 5 per cent to 50 per cent for the eighth floor from the top and for all lower floors. The Joint Code does not prescribe any load reductions, since this would be a general ordinance provision applying to concrete in common with all other types of building construction. Allowance must be made in case other load reduction provisions are in force.

(6) The size of columns will affect the quantities markedly. Allowance must be made if very small columns are required. In these designs the diameter or side of the top-story column has been made about one-twelfth of the panel dimension, and the percentage of column vertical reinforcement has been kept in the 2 to 3 per cent region generally. Stronger concrete has been used as the load increases, rather than very high steel percentages.

(7) If piles or caissons are required, the cost will be considerably

increased, except where piles are shown in the tables. Where piles are shown allowance must be made if the depth of pile cut-off requires additional excavation, unless gravity footings are possible and effect a balancing saving.

(8) In interpolating between the panel sizes and loadings covered by my computations the *trend* of the data as given in a diagram similar to Fig. 18 drawn with the proper unit prices for the work in hand, should be carefully studied.

(9) Joist and girder type, or beam and girder with other than third point beam spacing, will affect the quantities, and proper allowance should be made for the actual framing to be used.

(10) If the flat slab capital or depressed panel is to be different from the values given in Diagrams 77, 79, etc., in Part I of this paper, allowance must be made for the effect of such variation in increasing or decreasing quantities.

TABLE 144.—COST IN CENTS PER SQUARE FOOT OF TOTAL FLOOR AREA.

Side of Square Panel	Live Load, lb.	Soil Load, lb.	Concrete Strength, lb. per sq. in.	Flat Slab			Beam and Girder		
				6 Story	9 Story	12 Story	6 Story	9 Story	12 Story
18 ft. 0 in.	100	3000	2000	72.7	76.4	79.7	80.6	83.8	87.6
18 ft. 0 in	100	3000	3000	70.2	72.7	75.5	78.1	80.7	83.6
			Saving	**2.5**	**3.7**	**4.2**	**2.5**	**3.1**	**4.0**
18 ft. 0 in.	100	6000	2000	69.4	71.9	74.0	77.8	80.5	83.3
18 ft. 0 in.	100	6000	3000	67.6	69.2	70.7	75.2	77.8	79.6
			Saving	**1.8**	**2.7**	**3.3**	**2.6**	**2.7**	**3.7**
18 ft. 0 in.	300	3000	2000	94.7	102.1	108.7	110.0	119.0	126.4
18 ft. 0 in.	300	3000	3000	89.9	96.0	101.2	103.4	110.5	117.3
			Saving	**4.8**	**6.1**	**7.5**	**6 6**	**8.5**	**9.1**
18 ft. 0 in.	300	6000	2000	84.9	90.7	95.6	101.1	108.5	114.6
18 ft. 0 in.	300	6000	3000	81.4	86.2	89.7	96.0	101.6	106.6
			Saving	**3.5**	**4.5**	**5.9**	**5.1**	**6.9**	**8.0**
22 ft. 6 in.	100	3000	2000	82.6	87.2	91.5	86.0	90.4	94.2
22 ft. 6 in.	100	3000	3000	79.0	82.5	86.6	82.7	85.5	88.6
			Saving	**3.6**	**4.7**	**4.9**	**3.3**	**4.9**	**5.6**
22 ft. 6 in.	100	6000	2000	76.7	80.2	83.4	82.2	85.3	88.5
22 ft. 6 in.	100	6000	3000	74.5	77.1	79.8	79.4	81.5	83.9
			Saving	**2.2**	**3.1**	**3.6**	**2.8**	**3.8**	**4.6**
22 ft. 6 in.	300	3000	2000	110.2	125.7	131.8	116.8	136.1	143.9
22 ft. 6 in.	300	3000	3000	103.4	118.9	122.6	111.1	130.2	136.2
			Saving	**6.8**	**6.8**	**9.2**	**5.7**	**5.9**	**7.7**
22 ft. 6 in.	300	6000	2000	96.5	103.2	109.3	105.6	113.2	120.0
22 ft. 6 in.	300	6000	3000	91.4	97.5	101.3	101.3	107.6	113.1
			Saving	**5.1**	**5.7**	**8.0**	**4.3**	**5.6**	**6.9**

TABLE 145.—FLAT SLAB QUANTITIES—18 FT. BY 18 FT. INTERIOR PANEL. USUAL DESIGN—2000-LB. CONCRETE, EXCEPT IN COLUMNS.

Item No.	Item	Concrete		Reinforcing Steel		Formwork		Hand Excavation, cu. ft.
		2000-lb., cu. ft.	3000-lb., cu. ft.	Bars, lb.	Spiral, lb.	Wood, sq. ft.	Steel, units	
1	Roof	155	..	381	..	330	..	..
	100-LB. LIVE LOAD ON ALL FLOORS.							
2	Floor	193	..	506	..	331	..	..
3	1-6 Story Columns	113	64	1266	439	..	6	..
4	3000-lb. Footing	161	7	591	..	44	..	329
5	6000-lb. Footing	82	3	350	..	28	..	144
6	1-9 Story Columns	113	180	2742	804	..	9	..
7	3000-lb. Footing	293	15	1156	..	71	..	643
8	6000-lb. Footing	141	4	595	..	43	..	250
9	1-12 Story Columns	113	314	4868	1288	..	12	..
10	3000-lb. Footing	440	27	1820	..	90	..	1078
11	6000-lb. Footing	205	4	946	8	48	..	360
	300-LB. LIVE LOAD ON ALL FLOORS.							
12	Floor	236	..	855	..	333	..	..
13	1-6 Story Columns	57	144	2458	641	..	6	..
14	3000-lb. Footing	401	24	1695	..	82	..	960
15	6000-lb. Footing	198	5	883	..	44	..	338
16	1-9 Story Columns	57	300	5538	1294	..	9	..
17	3000-lb. Footing	687	47	2970	..	122	..	1804
18	6000-lb. Footing	339	8	1593	15	62	..	616
19	1-12 Story Columns	57	491	9668	2156	..	12	..
20	3000-lb. Footing	1028	76	4472	..	157	..	2740
21	6000-lb. Footing	507	12	2260	20	81	..	941

TABLE 146.—FLAT SLAB QUANTITIES—18 FT. BY 18 FT. INTERIOR PANEL. DESIGN USING 3000-LB. CONCRETE, EXCEPT IN COLUMNS.

Item No.	Item	Concrete			Reinforcing Steel		Formwork		Hand Excavation, cu. ft.
		2000-lb., cu. ft.	3000-lb., cu. ft.	5000-lb., cu. ft.	Bars, lb.	Spiral, lb.	Wood, sq. ft.	Steel, units	
22	Roof	..	136	..	407	..	329	..	..
	100-LB. LIVE LOAD ON ALL FLOORS.								
23	Floor	..	170	..	552	..	330	..	..
24	1–6 Story Columns	85	..	92	870	361	..	6	..
25	3000-lb. Footing	..	133	..	692	..	38	..	261
26	6000-lb. Footing	..	70	..	451	..	25	..	117
27	1–9 Story Columns	85	..	208	1733	604	..	9	..
28	3000-lb. Footing	..	224	..	1240	..	55	..	473
29	6000-lb. Footing	..	112	..	629	..	32	..	194
30	1–12 Story Columns	85	..	352	2910	925	..	12	..
31	3000-lb. Footing	..	337	..	2063	..	77	..	816
32	6000-lb. Footing	..	189	..	933	10	40	..	280
	300-LB. LIVE LOAD ON ALL FLOORS.								
33	Floor	..	204	..	992	..	332	..	..
34	1–6 Story Columns	57	28	116	1752	512	..	6	..
35	3000-lb. Footing	..	372	..	1782	..	77	..	740
36	6000-lb. Footing	..	148	4	855	..	39	..	256
37	1–9 Story Columns	57	28	272	3568	938	..	9	..
38	3000-lb. Footing	..	591	41	3316	..	97	..	1390
39	6000-lb. Footing	..	271	6	1569	12	53	..	475
40	1–12 Story Columns	57	28	463	6041	1487	..	12	..
41	3000-lb. Footing	..	787	67	5018	..	123	..	2166
42	6000-lb. Footing	..	391	8	2291	15	67	..	698

TABLE 147.—FLAT SLAB QUANTITIES—22 FT. 6 IN. BY 22 FT. 6 IN. INTERIOR PANEL.

USUAL DESIGN—2000 LB. CONCRETE, EXCEPT IN COLUMNS.

Item No.	Item	Concrete		Reinforcing Steel		Formwork		Hand Excavation, cu. ft.	Number of Concrete Piles
		2000-lb., cu. ft.	3000-lb., cu. ft.	Bars, lb.	Spiral, lb.	Wood, sq. ft.	Steel, units		
43	Roof	303	..	810	..	516	..	..	..
	100-LB. LIVE LOAD ON ALL FLOORS.								
44	Floor	378	..	1028	..	517	..	..	..
45	1-6 Story Columns	197	108	2282	600	..	6	..	..
46	3000-lb. Footing	365	20	1490	..	76	..	842	..
47	6000-lb. Footing	173	4	624	..	43	..	299	..
48	1-9 Story Columns	197	295	5144	1208	..	9	..	..
49	3000-lb. Footing	658	40	2784	..	109	..	1612	..
50	6000-lb. Footing	313	5	1453	11	60	..	535	..
51	1-12 Story Columns	197	503	9216	2046	..	12	..	..
52	3000-lb. Footing	987	71	4250	..	153	..	2620	..
53	6000-lb. Footing	474	16	2216	25	83	..	925	..
	300-LB. LIVE LOAD ON ALL FLOORS.								
54	Floor	464	..	1701	..	520	..	..	..
55	1-6 Story Columns	98	236	4279	954	..	6	..	..
56	3000-lb. Footing	852	58	3634	..	134	..	2210	..
57	6000-lb. Footing	421	9	1894	15	70	..	780	..
58	1-9 Story Columns	98	470	9831	2054	..	9	..	..
59	3000-lb. Footing	500	25	2540	50	115	..	1005	28
60	6000-lb. Footing	716	20	3300	65	104	..	1452	..
61	1-12 Story Columns	98	760	17291	3509	..	12	..	..
62	3000-lb. Footing	825	29	3493	110	147	..	1560	38
63	6000-lb. Footing	1086	35	4940	130	141	..	2252	..

TABLE 148.—FLAT SLAB QUANTITIES—22 FT. 6 IN. BY 22 FT. 6 IN. INTERIOR PANEL.

DESIGN USING 3000-LB. CONCRETE, EXCEPT IN COLUMNS.

Item No.	Item	Concrete			Reinforcing Steel		Formwork		Hand Excavation, cu. ft.	Number of Concrete Piles
		2000-lb., cu. ft.	3000-lb., cu. ft.	5000-lb., cu. ft.	Bars, lb.	Spiral, lb.	Wood, sq. ft.	Steel, units		
64	Roof	..	264	..	882	..	514	..	..	..
	100-LB. LIVE LOAD ON ALL FLOORS.									
65	Floor	..	330	..	1135	..	516	..	..	..
66	1–6 Story Columns	197	..	108	1826	536	..	6	..	..
67	3000-lb. Footing	..	287	..	1640	..	69	..	601	..
68	6000-lb. Footing	..	140	..	798	..	39	..	236	..
69	1–9 Story Columns	197	..	295	3721	977	..	9	..	..
70	3000-lb. Footing	..	499	..	3026	..	87	..	1153	..
71	6000-lb. Footing	..	246	5	1480	..	50	..	420	..
72	1–12 Story Columns	197	..	503	6107	1495	..	12	..	..
73	3000-lb. Footing	..	749	72	4957	..	129	..	2160	..
74	6000-lb. Footing	..	344	9	2045	..	65	..	621	..
	300-LB. LIVE LOAD ON ALL FLOORS.									
75	Floor	..	396	..	1942	..	518	..	..	..
76	1–6 Story Columns	98	49	187	3008	735	..	6	..	..
77	3000-lb. Footing	..	694	..	3979	..	114	..	1725	..
78	6000-lb. Footing	..	312	8	1866	15	62	..	574	..
79	1–9 Story Columns	98	49	421	6458	1445	..	9	..	..
80	3000-lb. Footing	..	450	22	2400	35	106	..	881	26
81	6000-lb. Footing	..	545	18	3303	46	89	..	1122	..
82	1–12 Story Columns	98	49	710	11051	2376	..	12	..	..
83	3000-lb. Footing	..	660	11	2328	32	119	..	1144	36
84	6000-lb. Footing	..	825	30	4909	78	119	..	1679	..

TABLE 149.—BEAM AND GIRDER QUANTITIES—18 FT. BY 18 FT. INTERIOR PANEL.

USUAL DESIGN—2000-LB. CONCRETE, EXCEPT IN COLUMNS.

Item No.	Item	Concrete		Reinforcing Steel		Wood Forms, sq. ft.	Hand Excavation, cu. ft.
		2000-lb., cu. ft.	3000-lb., cu. ft.	Bars, lb.	Spiral, lb.		
1	Roof	140	..	563	53	471	..
	100-LB. LIVE LOAD ON ALL FLOORS.						
2	Floor	147	..	738	79	471	..
3	1–6 Story Columns	75	87	1031	408	411	..
4	3000-lb. Footing	144	..	489	..	39	255
5	6000-lb. Footing	70	..	275	..	26	115
6	1–9 Story Columns	75	212	2268	738	669	..
7	3000-lb. Footing	244	..	870	..	55	484
8	6000-lb. Footing	117	4	465	..	35	206
9	1–12 Story Columns	75	368	4112	1190	957	..
10	3000-lb. Footing	362	20	1421	..	76	844
11	6000-lb. Footing	173	4	798	..	43	304
	300-LB. LIVE LOAD ON ALL FLOORS.						
12	Floor	190	..	1136	190	541	..
13	1–6 Story Columns	50	159	2500	685	480	..
14	3000-lb. Footing	362	20	1488	..	76	845
15	6000-lb. Footing	168	4	792	..	43	294
16	1–9 Story Columns	50	340	5680	1404	798	..
17	3000-lb. Footing	643	41	2699	..	108	1594
18	6000-lb. Footing	298	13	1429	..	65	613
19	1–12 Story Columns	50	568	10020	2356	1155	..
20	3000-lb. Footing	948	67	4078	..	142	2500
21	6000-lb. Footing	448	34	2125	..	95	1045

TABLE 150.—BEAM AND GIRDER QUANTITIES—18 FT. BY 18 FT. INTERIOR PANEL.

DESIGN USING 3000-LB. CONCRETE, EXCEPT IN COLUMNS.

Item No.	Item	Concrete			Reinforcing Steel		Wood Forms, sq. ft.	Hand Excavation, cu. ft.
		2000-lb., cu. ft.	3000-lb., cu. ft.	5000-lb., cu. ft.	Bars, lb.	Spiral, lb.		
22	Roof	..	127	..	466	26	476	..
	100-LB. LIVE LOAD ON ALL FLOORS.							
23	Floor	..	135	..	635	51	475	..
24	1–6 Story Columns	75	25	62	872	379	411	..
25	3000-lb. Footing	..	114	..	556	..	34	202
26	6000-lb. Footing	..	59	..	298	..	24	98
27	1–9 Story Columns	75	25	188	1474	585	699	..
28	3000-lb. Footing	..	197	..	1008	..	47	356
29	6000-lb. Footing	..	102	..	483	..	32	171
30	1–12 Story Columns	75	25	343	2426	852	957	..
31	3000-lb. Footing	..	296	..	1631	..	64	664
32	6000-lb. Footing	..	126	..	762	..	37	236
	300-LB. LIVE LOAD ON ALL FLOORS							
33	Floor	..	169	..	925	152	548	..
34	1–6 Story Columns	50	..	159	1727	540	480	..
35	3000-lb. Footing	..	296	..	1699	..	64	660
36	6000-lb. Footing	..	145	..	786	..	39	242
37	1–9 Story Columns	50	..	340	3720	1026	798	..
38	3000-lb. Footing	..	512	..	2984	..	93	1250
39	6000-lb. Footing	..	246	5	1405	..	51	421
40	1–12 Story Columns	50	..	568	6590	1667	1155	..
41	3000-lb. Footing	..	775	..	4802	..	124	2000
42	6000-lb. Footing	..	352	9	2285	..	65	651

TABLE 151.—BEAM AND GIRDER QUANTITIES—22 FT. 6 IN. BY 22 FT. 6 IN. INTERIOR PANEL.

USUAL DESIGN—2000-LB. CONCRETE, EXCEPT IN COLUMNS.

Item No.	Item	Concrete		Reinforcing Steel		Wood Forms, sq. ft.	Hand Excavation, cu. ft.	Number of Concrete Piles
		2000-lb., cu. ft.	3000-lb., cu. ft.	Bars, lb.	Spiral, lb.			
43	Roof	230	..	1043	81	739	..	..
	100-LB. LIVE LOAD ON ALL FLOORS.							
44	Floor	250	..	1219	143	796	..	..
45	1–6 Story Columns	114	131	1869	573	524	..	..
46	3000-lb. Footing	266	13	1017	..	60	574	..
47	6000-lb. Footing	131	4	588	..	36	230	..
48	1–9 Story Columns	114	313	4241	1138	843	..	..
49	3000-lb. Footing	485	26	1976	..	88	1120	..
50	6000-lb. Footing	222	5	1037	..	50	388	..
51	1–12 Story Columns	114	543	7630	1893	1201	..	..
52	3000-lb. Footing	722	47	2986	..	119	1810	..
53	6000-lb. Footing	348	7	1603	..	68	614	..
	300-LB. LIVE LOAD ON ALL FLOORS.							
54	Floor	311	..	2208	340	846	..	..
55	1–6 Story Columns	78	228	4144	1015	587	..	..
56	3000-lb. Footing	685	48	2953	..	116	1793	..
57	6000-lb. Footing	330	10	1585	..	64	600	..
58	1–9 Story Columns	78	496	9425	2153	979	..	..
59	3000-lb. Footing	547	62	2135	..	225	953	25
60	6000-lb. Footing	590	20	2710	..	90	1166	..
61	1–12 Story Columns	78	857	16438	3654	1435	..	..
62	3000-lb. Footing	787	243	3365	..	340	1798	33
63	6000-lb. Footing	903	35	4096	..	120	1830	..

TABLE 152.—BEAM AND GIRDER QUANTITIES—22 FT. 6 IN. BY 22 FT. 6 IN. INTERIOR PANEL.

DESIGN USING 3000-LB. CONCRETE, EXCEPT IN COLUMNS.

Item No.	Item	Concrete			Reinforcing Steel		Wood Forms, sq. ft.	Hand Excavation, cu. ft.	Number of Concrete Piles
		2000-lb., cu. ft.	3000-lb., cu. ft.	5000-lb., cu. ft.	Bars, lb.	Spiral, lb.			
64	Roof	..	207	..	846	59	747	..	..
	100-LB. LIVE LOAD ON ALL FLOORS.								
65	Floor	..	220	..	1092	96	804	..	..
66	1–6 Story Columns	114	38	93	1564	508	524	..	..
67	3000-lb. Footing	..	219	..	1206	..	52	435	..
68	6000-lb. Footing	..	110	..	597	..	33	184	..
69	1–9 Story Columns	114	38	275	2755	840	843	..	..
70	3000-lb. Footing	..	378	..	2042	..	75	832	..
71	6000-lb. Footing	..	182	..	966	..	42	305	..
72	1–12 Story Columns	114	38	505	4574	1291	1201	..	..
73	3000-lb. Footing	..	569	..	3183	..	102	1372	..
74	6000-lb. Footing	..	269	..	1547	..	53	452	..
	300-LB. LIVE LOAD ON ALL FLOORS.								
75	Floor	..	289	..	1931	283	855	..	..
76	1–6 Story Columns	78	41	188	3216	804	587	..	..
77	3000-lb. Footing	..	572	..	3396	..	99	1431	..
78	6000-lb. Footing	..	268	6	1596	..	54	470	..
79	1–9 Story Columns	78	41	455	6688	1584	979	..	..
80	3000-lb. Footing	..	532	..	2048	..	200	866	24
81	6000-lb. Footing	..	464	13	2830	..	80	888	..
82	1–12 Story Columns	78	41	816	11261	2504	1435	..	..
83	3000-lb. Footing	..	821	..	3135	..	295	1535	32
84	6000-lb. Footing	..	706	22	4252	..	106	1416	..

TABLE 153.—FLAT SLAB QUANTITIES—18 FT. BY 18 FT. INTERIOR PANEL. TOTAL QUANTITIES FOR ONE PANEL FROM FOOTING TO ROOF.

Item No.	Number of Stories	Live Load, lb. per sq. ft.	Soil Load, lb. per sq. ft.	Concrete, lb. per sq. in.	Concrete			Reinforcing Steel		Formwork		Hand Excavation, cu. ft.
					2000-lb., cu. ft.	3000-lb., cu. ft.	5000-lb., cu. ft.	Bars, lb.	Spiral, lb.	Wood, sq. ft.	Steel, units	
101	6	100	3000	2000	1394	71	..	4768	439	2029	6	329
102	6	100	3000	3000	85	1119	92	4729	361	2017	6	261
103	9	100	3000	2000	2105	195	..	8327	804	3049	9	643
104	9	100	3000	3000	85	1720	208	7796	604	3024	9	473
105	12	100	3000	2000	2831	341	..	12635	1288	4061	12	1078
106	12	100	3000	3000	85	2343	352	11452	925	4036	12	816
107	6	100	6000	2000	1315	67	..	4537	439	2013	6	144
108	6	100	6000	3000	85	1056	92	4488	361	2004	6	117
109	9	100	6000	2000	1953	184	..	7781	804	3021	9	250
110	9	100	6000	3000	85	1608	208	7185	604	3001	9	194
111	12	100	6000	2000	2596	318	..	11761	1296	4019	12	360
112	12	100	6000	3000	85	2173	352	10260	911	3999	12	280
113	6	300	3000	2000	1793	168	..	8809	641	2077	6	960
114	6	300	3000	3000	57	1506	116	8937	512	2066	6	740
115	9	300	3000	2000	2787	347	..	15729	1294	3116	9	1760
116	9	300	3000	3000	57	2321	313	15227	938	3082	9	1390
117	12	300	3000	2000	3836	567	..	23899	2156	4150	12	2740
118	12	300	3000	3000	57	3195	530	22378	1487	4104	12	2166
119	6	300	6000	2000	1590	149	..	8004	641	2039	6	338
120	6	300	6000	3000	57	1332	120	8010	512	2028	6	256
121	9	300	6000	2000	2439	308	..	14352	1309	3056	9	616
122	9	300	6000	3000	57	2067	278	13408	950	3038	9	475
123	12	300	6000	2000	3315	503	..	21741	2176	4074	12	941
124	12	300	6000	3000	57	2799	471	19651	1502	4048	12	698

TABLE 154.—FLAT SLAB QUANTITIES—22 FT. 6 IN. BY 22 FT. 6 IN. INTERIOR PANEL.

TOTAL QUANTITIES FOR ONE PANEL FROM FOOTING TO ROOF.

Item No.	Number of Stories	Live Load, lb. per sq. ft.	Soil Load, lb. per sq. ft.	Concrete, lb. per sq. in.	Concrete			Reinforcing Steel		Formwork		Hand Excavation, cu. ft.	Number of Concrete Piles
					2000-lb., cu. ft.	3000-lb., cu. ft.	5000-lb., cu. ft.	Bars, lb.	Spiral, lb.	Wood, sq. ft.	Steel, units		
125	6	100	3000	2000	2755	128	..	9722	600	3177	6	842	..
126	6	100	3000	3000	197	2201	108	10017	536	3163	6	601	..
127	9	100	3000	2000	4182	335	..	16862	1208	4761	9	1612	..
128	9	100	3000	3000	197	3403	295	16709	977	4729	9	1153	..
129	12	100	3000	2000	5646	574	..	25584	2154	6356	12	2620	..
130	12	100	3000	3000	197	4643	575	24431	1495	6319	12	2160	..
131	6	100	6000	2000	2563	112	..	8856	600	3144	6	299	..
132	6	100	6000	3000	197	2054	108	9175	536	3133	6	236	..
133	9	100	6000	2000	3837	300	..	15631	1219	4712	9	535	..
134	9	100	6000	3000	197	3150	300	15163	977	4692	9	420	..
135	12	100	6000	2000	5132	519	..	23550	2071	6286	12	925	..
136	12	100	6000	3000	197	4238	512	21519	1495	6255	12	621	..
137	6	300	3000	2000	3573	294	..	17228	954	3250	6	2210	..
138	6	300	3000	3000	98	2987	187	17579	735	3218	6	1725	..
139	9	300	3000	2000	4613	495	..	26789	2104	4791	9	1005	28
140	9	300	3000	3000	98	3931	443	25276	1490	4764	9	881	26
141	12	300	3000	2000	6330	789	..	40305	3619	6383	12	1560	38
142	12	300	3000	3000	98	5329	721	35623	2408	6331	12	1144	36
143	6	300	6000	2000	3142	245	..	15488	969	3186	6	780	..
144	6	300	6000	3000	98	2605	195	15460	750	3166	6	574	..
145	9	300	6000	2000	4829	490	..	27509	2119	4780	9	1452	..
146	9	300	6000	3000	98	4026	439	26151	1498	4747	9	1122	..
147	12	300	6000	2000	6581	795	..	41752	3639	6377	12	2252	..
148	12	300	6000	3000	98	5486	740	38204	2454	6326	12	1679	..

TABLE 155.—BEAM AND GIRDER QUANTITIES—18 FT. BY 18 FT. INTERIOR PANEL.

TOTAL QUANTITIES FOR ONE PANEL FROM FOOTING TO ROOF.

Item No.	Number of Stories	Live Load, lb. per sq. ft.	Soil Load, lb. per sq. ft.	Concrete, lb. per sq. in.	Concrete			Reinforcing Steel		Wood Forms, sq. ft.	Hand Excavation, cu. ft.
					2000-lb., cu. ft.	3000-lb., cu. ft.	5000-lb., cu. ft.	Bars, lb.	Spiral, lb.		
101	6	100	3000	2000	1096	87	..	5773	856	3276	255
102	6	100	3000	3000	75	939	62	5069	660	3296	202
103	9	100	3000	2000	1637	212	..	9605	1423	4963	484
104	9	100	3000	3000	75	1426	188	8128	1019	4992	356
105	12	100	3000	2000	2197	388	..	14214	2112	6685	844
106	12	100	3000	3000	75	1929	343	11508	1439	6722	664
107	6	100	6000	2000	1022	87	..	5559	856	3263	115
108	6	100	6000	3000	75	884	62	4811	660	3286	98
109	9	100	6000	2000	1510	216	..	9200	1423	4943	206
110	9	100	6000	3000	75	1331	188	7505	1019	4977	171
111	12	100	6000	2000	2008	372	..	13591	2112	6652	304
112	12	100	6000	3000	75	1759	343	10639	1473	6695	236
113	6	300	3000	2000	1502	179	..	10231	1688	3732	845
114	6	300	3000	3000	50	1268	159	8517	1326	3760	660
115	9	300	3000	2000	2353	381	..	18030	2977	5705	1594
116	9	300	3000	3000	50	1991	340	14570	2242	5751	1250
117	12	300	3000	2000	3228	635	..	27157	4499	7719	2500
118	12	300	3000	3000	50	2761	568	22033	3365	7763	2000
119	6	300	6000	2000	1308	163	..	9535	1688	3699	294
120	6	300	6000	3000	50	1117	159	7604	1326	3735	242
121	9	300	6000	2000	2008	353	..	16760	2977	5662	613
122	9	300	6000	3000	50	1725	345	12991	2242	5709	421
123	12	300	6000	2000	2728	602	..	25204	4499	7672	1045
124	12	300	6000	3000	50	2338	577	19516	3365	7704	651

TABLE 156.—BEAM AND GIRDER QUANTITIES—22 FT. 6 IN. BY 22 FT. 6 IN. INTERIOR PANEL.

TOTAL QUANTITIES FOR ONE PANEL FROM FOOTING TO ROOF.

Item No.	Number of Stories	Live Load, lb. per sq. ft.	Soil Load, lb. per sq. ft.	Concrete, lb. per sq. in.	Concrete			Reinforcing Steel		Wood Forms, sq. ft.	Hand Excavation, cu. ft.	Number of Concrete Piles
					2000-lb., cu. ft.	3000-lb., cu. ft.	5000-lb., cu. ft.	Bars, lb.	Spiral, lb.			
125	6	100	3000	2000	1860	144	..	10024	1369	5303	574	..
126	6	100	3000	3000	114	1564	93	9070	1037	5343	435	..
127	9	100	3000	2000	2829	339	..	17012	2363	8038	1120	..
128	9	100	3000	3000	114	2383	275	14379	1667	8081	832	..
129	12	100	3000	2000	3816	590	..	25068	3547	10815	1810	..
130	12	100	3000	3000	114	3234	505	20615	2406	10872	1372	..
131	6	100	6000	2000	1725	135	..	9595	1369	5279	230	..
132	6	100	6000	3000	114	1455	93	8467	1037	5314	184	..
133	9	100	6000	2000	2566	318	..	16073	2363	8000	388	..
134	9	100	6000	3000	114	2187	275	13303	1667	8048	305	..
135	12	100	6000	2000	3442	550	..	23685	3547	10764	614	..
136	12	100	6000	3000	114	2934	505	18979	2406	10845	452	..
137	6	300	3000	2000	2548	276	..	19180	2796	5672	1793	..
138	6	300	3000	3000	78	2265	198	17113	2278	5708	1431	..
139	9	300	3000	2000	3343	558	..	30267	4954	8711	953	25
140	9	300	3000	3000	78	3092	455	25030	3907	8766	866	24
141	12	300	3000	2000	4516	1100	..	45134	7475	11820	1798	33
142	12	300	3000	3000	78	4248	816	36483	5676	11882	1535	32
143	6	300	6000	2000	2193	238	..	17822	2796	5620	676	..
144	6	300	6000	3000	78	1961	194	15313	2278	5663	470	..
145	9	300	6000	2000	3386	516	..	30842	4954	8576	1166	..
146	9	300	6000	3000	78	3024	468	25812	3907	8648	888	..
147	12	300	6000	2000	4632	892	..	45865	7475	11600	1830	..
148	12	300	6000	3000	78	4133	838	37600	5676	11693	1416	..

JOINT CODE

Building Regulations for Reinforced Concrete

Report of Committee E-1 on Reinforced Concrete Building Design and Specifications amended and adopted as a tentative standard at the Twenty-Fourth Annual Convention of the American Concrete Institute, February 29, 1928, and adopted as a tentative standard by the Reinforcing Steel Institute at the Fourth Annual Meeting, March 19, 1928.

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INTRODUCTION.

These regulations have been prepared for use as part of a general building code. When so used, it is necessary that the following definitions, which give the meaning of certain terms as used in the regulations, become a part of the code. They should appear either in a general chapter in the code relating to definitions or in a chapter by themselves preceding these regulations for the use of reinforced concrete:

Definitions.

Aggregate.—Inert material which is mixed with portland cement and water to produce concrete; in general, aggregate consists of sand, pebbles, gravel, crushed stone, blast-furnace slag, or similar materials.

Anchorage.—The embedment in concrete of a portion of a reinforcement bar, either straight or with hooks, designed to prevent pulling out or slipping of the bar when subjected to stress. (The anchorage of tension reinforcement in beams includes only the embedded length beyond a point of contra-flexure or of zero moment.)

Blast-Furnace Slag.—The non-metallic product, consisting essentially of silicates and alumino-silicates of lime, which is developed simultaneously with iron in a blast furnace.

Column.—An upright compression member the length of which exceeds three times its least lateral dimension.

Column Capital.—An enlargement of the upper end of a reinforced-concrete column designed and built to act as a unit with the column and flat slab.

Column Strip.—A portion of a flat slab panel one-half panel in width occupying the two quarter-panel areas outside of the middle strip. (See Middle Strip.)

Add also definition as follows:

Combination Column.—A column in which a structural steel section, designed to carry the principal part of the load, is wrapped with wire and encased in concrete of such quality that some additional load may be allowed.

Composite Column.—A column in which a concrete core enclosed by spiral reinforcement and further reinforced by longitudinal bars encases a structural steel or cast iron column designed to carry a portion of the load.

Concrete.—A mixture of portland cement, fine aggregate, coarse aggregate and water. (See Mortar.)

Consistency.—A general term used to designate the relative plasticity of freshly mixed concrete or mortar.

Crushed Stone.—Bedded rock or boulders, which have been broken by mechanical means into fragments of varying shapes and sizes.

Dead-Load.—The weight of the permanent parts of the structure.

Deformed Bar.—Reinforcement bars with closely spaced shoulders, lugs or projections formed integrally with the bar during rolling so as to firmly engage the surrounding concrete. Wire mesh with welded intersections not farther apart than twelve inches in the direction of the principal reinforcing and with cross wires not smaller than No. 10 may be rated as a deformed bar.

Diagonal Band.—In a four-way flat slab system a group of bars covering a width approximately 0.4 the average span, symmetrical with respect to the diagonal running from corner to corner of the panel.

Diagonal Direction.—A direction parallel or approximately parallel to the diagonal of the panel of a flat slab.

Direct Band.—In a four-way flat slab system, a group of bars covering a width approximately 0.4 l_1, symmetrical with respect to the line of centers of supporting columns.

Dropped Panel.—The structural portion of a flat slab which is thickened throughout an area surrounding the column capital.

Effective Area of Concrete.—The area of a section which lies between the centroid of the tension reinforcement and the compression surface in a beam or slab, and having a width equal to the width of the rectangular beam or slab, or the effective width of the flange of a Tee beam.

Effective Area of Reinforcement.—The area obtained by multiplying the right cross-sectional area of the metal reinforcement by the cosine of the angle between its direction and that for which the effectiveness of the reinforcement is to be determined.

Flat Slab.—A reinforced-concrete slab generally without beams or girders to transfer the loads to supporting members.

Footing.—A structural unit used to distribute wall or column loads to the foundation materials.

Gravel.—Rounded particles larger than sand grains resulting from the natural disintegration of rocks. (See Sand.)

Laitance.—Extremely fine material of little or no hardness which may collect on the surface of freshly deposited concrete or mortar, resulting from the use of excess mixing water, usually recognized by its relatively light color.

Live-Load.—Loads and forces other than the dead-load.

Middle Strip.—A portion of a flat slab panel one-half panel in width, symmetrical with respect to the panel center line and extending through the panel in the direction in which moments are being considered.

Mortar.—A mixture of portland cement, fine aggregate, and water. (See Concrete.)

Negative Bending Moment.—That moment which exists between a support of a slab or beam and the point of inflection on either side of the support.

Negative Reinforcement.—Reinforcement so placed as to take tensile stress due to negative bending moment.

Paneled Ceiling.—A paneled ceiling refers to a flat slab in which approximately that portion of the area enclosed within the intersection of the two middle strips is reduced in thickness.

Panel Length.—The distance in either rectangular direction between centers of two columns of a panel.

Pedestal.—An upright compression member whose height does not exceed three times its least lateral dimension.

Pedestal Footing.—A column footing projecting less than one-half its depth from the faces of the column on all sides and having a depth not more than three times its least width.

Plain Concrete.—Concrete without metal reinforcement.

Portland Cement.—The product obtained by finely pulverizing clinker produced by calcining to incipient fusion an intimate and properly proportioned mixture of argillaceous and calcareous materials, with no additions subsequent to calcination excepting water and calcined or uncalcined gypsum.

Positive Bending Moment.—That moment which exists at all other points in beams or slabs except where negative moment exists.

Positive Reinforcement.—Reinforcement so placed as to take tensile stress due to positive bending moment.

Principal Design Section.—The vertical sections in a flat slab on which the moments in the rectangular directions are critical. (See Sec. 1002.)

Ratio of Reinforcement.—The ratio of the effective area of the reinforcement cut by a section of a beam or slab to the effective area of the concrete at that section.

Rectangular Direction.—A direction parallel to a side of the panel of a flat slab.

Reinforced Concrete.—Concrete in which metal is embedded in such a manner that the two materials act together in resisting forces.

Sand.—Small grains resulting from the natural disintegration of rocks. (See Gravel.)

Screen.—A metal plate with closely spaced circular perforations. (See Sieve.)

Sieve.—Woven wire cloth with square openings. (See Screen.)

Strut.—A compression member other than a column or pedestal.

Surface Water.—By the term "surface water' 'is meant all water carried by the aggregate except that held within the aggregate particles themselves by absorption.

Wall Beam.—A reinforced-concrete beam which extends from column to column along the outer edge of a wall panel.

Water-Cement Ratio.—By the water-cement ratio is meant the total quantity of water entering the mixture including the surface water carried by the aggregate, expressed in terms of the quantity of cement. The water-cement ratio shall be expressed in U. S. gallons per sack (94 lb.) of cement.

Tentative Building Regulations for Reinforced Concrete.*

CHAPTER 1.

General.

101: *Scope:*

(a) These regulations cover the use of reinforced concrete in any structure to be erected under the provisions of the building code of which they form a part. They are intended to supplement the general provisions of the code in order to provide for the proper design and construction of structures of this material. In all matters pertaining to the design and construction where these specific regulations are in conflict with other provisions of the code, these regulations shall govern.

102: *Permits and Drawings:*

(a) Drawings and typical details of all reinforced-concrete construction showing the sizes and position of all structural members, metal reinforcement, and the live-load used in the design shall be filed with the department as a permanent record before a permit to construct such work shall be issued. All calculations made may be required by the department to be submitted with the drawings.

103: *Special Systems of Reinforced Concrete:*

(a) The sponsors of any system of reinforced concrete which has been in successful use, or the adequacy of which has been shown by test, and the design of which is either in conflict with these provisions or not covered by them, shall have the right to present the data on which their design is based to a "Board of Examiners for Special Construction." This Board shall be composed of competent engineers, architects and builders. The Board shall have the power to investigate the data so submitted and to formulate rulings governing the design and construction of such systems, which ruling shall be of the same force and effect as the provisions of this code. This Board is to be designated as provided elsewhere in the code.

CHAPTER 2.

Materials and Tests.

201: *Tests:*

(a) The tests called for in these regulations when ordered in accordance with the provisions of this chapter by the commissioner of buildings or his authorized representatives shall be arranged for by the owner or his representative. No responsibility for the expense of these tests shall attach to the department of buildings. Such tests shall be made in accordance with the standard method of test covering the particular material under consideration, of the American Society for Testing Materials in effect on the date of the adoption of these regulations, except as noted herein.

(b) All such tests shall be made by competent persons. The com-

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* The report of Committee E-1, Reinforced-Concrete Building Design and Specifications, carrying proposed building regulations for reinforced concrete is here published as amended on the floor of the convention, Feb. 29, 1928. As amended this report was adopted as Tentative Building Regulations for Reinforced Concrete (E-1A-28T).

petency of the persons making the tests shall be judged by their training and experience. The commissioner of buildings may disapprove for just cause those whose records show technical incompetency. Copies of the results of all tests shall be kept on file in the office of the commissioner of buildings for a period of two years after the acceptance of the structure. Tests shall be made on any material entering into concrete or reinforced-concrete construction when there is reasonable doubt as to its suitability for the purpose.

(c) The commissioner of buildings or his authorized representative shall have the right to require reasonable tests of the concrete from time to time to determine whether the materials and methods in use are such as to produce concrete of the necessary quality. Specimens for such tests shall be taken at the place where concrete is being deposited, and shall be taken, cured, and tested in accordance with the "Standard Method of Making Compression Tests of Concrete" (Serial Designation: C 39-27) of the American Society for Testing Materials.

202: *Load Tests:*

(a) The commissioner of buildings or his authorized representative shall have the right to order the test under load of any portion of a completed structure, when the conditions have been such as to leave reasonable doubt as to the adequacy of the structure to serve the purpose for which it was intended. Such tests shall not be required to be made on any concrete construction which is less than 60 days old.

(b) In such tests, the member or portion of the structure under consideration shall be subject to a superimposed load equal to one and one-half times the live load plus one-half of the dead load. This load shall be left in position for a period of twenty-four hours before removal. If, during the test, or upon removal of the load, the member or portion of structure shows evident failure, such changes or modifications as are necessary to make the structure adequate for the rated capacity shall be made, or where lawful, a lower rating shall be established. The structure will be considered to have failed to pass the test if within twenty-four hours after the removal of the load the slabs or beams do not show a recovery of at least 75 per cent of the maximum deflection shown during the twenty-four hours while under load.

203: *Inspection:*

(a) All concrete work shall be inspected by the architect or engineer responsible for its design or by a competent representative responsible to the architect or the engineer. A record shall be kept of such inspection which shall cover the quality and quantity of concrete materials, including water, the mixing and placing of the concrete, and the placing of the reinforcing steel. The inspection record shall also include a complete record of the progress of the work and of the temperatures, when these fall below 40 deg. F., and of the protection given to the concrete while curing. These records shall be available for inspection by the commis-

sioner of buildings at all times during the progress of the work and shall be preserved for two (2) years after the acceptance of the structure.

204: *Portland Cement:*

(a) Portland cement shall conform to the "Standard Specifications and Tests for Portland Cement". (Serial Designation: C 9-26) of the American Society for Testing Materials.

205: *Concrete Aggregates:*

(a) Concrete aggregates shall consist of natural sands and gravels, crushed rock, crushed air-cooled blast-furnace slag, or other inert materials having clean, uncoated grains of strong and durable minerals. Aggregates containing soft, friable, thin, flaky, elongated, or laminated particles totaling more than 3 per cent, or containing shale in excess of 1½ per cent, or silt and crusher dust finer than the No. 100 standard sieve in excess of 2 per cent shall not be used. These percentages shall be based on the weight of the combined aggregate as used in the concrete. When all three groups of these deleterious materials are present in the aggregates, the combined amounts shall not exceed 5 per cent by weight of the combined aggregate.

(b) Aggregates shall not contain strong alkali or organic material which gives a color darker than the standard color when tested in accordance with the "Standard Method of Test for Organic Impurities in Sands for Concrete" (Serial Designation: C 40-27) of the American Society for Testing Materials.

(c) The maximum size of the aggregate shall be not larger than one-fifth of the narrowest dimension between forms of the member for which the concrete is to be used nor larger than three-fourths of the minimum clear spacing between reinforcing bars. By maximum size of aggregate is meant the clear space between the sides of the smallest square opening through which 95 per cent by weight of the material can be passed.

206: *Water:*

(a) Water used in mixing concrete shall be clean, and free from strong acids, alkalis, or organic materials.

207: *Metal Reinforcement:*

(a) Metal reinforcement shall conform to the requirements of the "Standard Specifications for Billet-Steel Concrete Reinforcement Bars" of Intermediate Grade[1] (Serial Designation: A 15-14), or for "Rail-Steel Concrete Reinforcement Bars" (Serial Designation: A 16-14) of the American Society for Testing Materials. The provision in these specifications for machining deformed bars before testing shall be eliminated.

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[1] This recommendation is in accordance with "Commercial Standard No. 1 (New Billet-Steel Concrete Reinforcing Bars)" of the U. S. Department of Commerce, which establishes the intermediate grade as the single standard for billet-steel reinforcement. Until such time as existing stocks of structural and hard-grade billet-steel reinforcement, meeting the requirements of A. S. T. M. Specification A 15-14 are exhausted, these grades may be used with the unit stresses specified in Sec. 307.

(b) Wire for concrete reinforcement shall conform to the requirements of the "Standard Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement" (Serial Designation: A 82-27) of the American Society for Testing Materials.

(c) Structural steel shall conform to the requirements of the "Standard Specifications for Structural Steel for Buildings" (Serial Designation: A9-24) of the American Society for Testing Materials.

(d) Cast-iron sections for composite or combination columns shall conform to "Standard Specifications for Cast-Iron Pipe and Special Castings" (Serial Designation: A44-04) of the American Society for Testing Materials.

208: *Storage of Materials:*

(a) Cement and aggregates shall be stored at the work in a manner to prevent deterioration or the intrusion of foreign matter. Any material which has deteriorated or has been damaged shall be immediately and completely removed from the work.

CHAPTER 3.

CONCRETE QUALITY AND WORKING STRESSES.

301: *Concrete Quality:*

(a) The working stresses for the design of reinforced-concrete structures shall be based upon the minimum ultimate 28-day strength of the concrete to be used in the structure in accordance with the values given in Sec. 306. All plans submitted for approval or used on the work shall clearly show the strength of concrete for which all parts of the structure were designed. The strength of concrete shall be fixed in terms of the water-cement ratio in accordance with one of the following methods:

(1) By established results for average materials, as provided in Sec. 302.

(2) By specific test of materials for the structure, as provided in Sec. 303.

(b) By the water-cement ratio is meant the total quantity of water entering the mixture including the surface water carried by the aggregate, expressed in terms of the quantity of cement. The water-cement ratio shall be expressed in U. S. gallons per sack (94 lb.) of cement.

302: *Water-Cement Ratio for Average Materials:*

(a) Where no preliminary tests of the materials to be used are made, the water-cement ratios shall not exceed the values in the following table. The mixes shown in the table are approximate only, and may require adjustment to give proper workability.

ASSUMED STRENGTH OF CONCRETE MIXTURES

Water-Cement Ratio U. S. gallons per 94-lb. sack of cement	Approximate Mix Volume of Portland Cement to Sum of Separate Volumes of Fine and Coarse Aggregate as Measured Dry Plastic Concrete	Assumed Compressive Strength at 28 days in pounds per square inch
8¼	1 : 7	1,500
7½	1 : 6	2,000
6¾	1 : 5¼	2,500
6	1 : 4½	3,000
	Moderately Wet Concrete	
8¼	1 : 6½	1,500
7½	1 : 5½	2,000
6¾	1 : 4¾	2,500
6	1 : 4	3,000

NOTE: In interpreting this table, surface water contained in the aggregate must be included as part of the mixing water in computing the water-cement ratio.

(b) During the progress of the work, a reasonable number of compression tests shall be made as may be required by the commissioner of buildings, but at least one specimen shall be tested for each 100 cu. yd. of concrete being placed. The tests shall be made in accordance with provisions of Sec. 304. Should the average 28-day strength fall below the minimum ultimate strength called for on the plans, the commissioner of buildings shall have the right to require a load test under the provisions of Sec. 202.

303: *Water-Cement Ratio by Tests of Materials:*

(a) Where the water-cement ratios for the various strengths of concrete are to be established by test, these tests shall be made in advance of the beginning of operations using the materials proposed and consistencies suitable for the work and in accordance with the "Standard Method of Making Compression Tests of Concrete" (Serial Designation C 39-27) of the American Society for Testing Materials, including the provisions for curing in a moist room at 70 deg. F. and testing wet. A curve representing the relation between the average 28-day strength of the concrete and water-cement ratio shall be established for a range of values including all of the strengths called for in the plans. The tests shall include at least four different water-cement ratios and at least four specimens for each water-cement ratio. The water-cement ratio to be used in the structure shall be that corresponding to a point on the curve established by these tests representing a strength of concrete 15 per cent higher than the minimum ultimate strength called for on the plans and satisfactory evidence shall be submitted to show that these water-cement ratios are not exceeded. No substitution shall be made in the materials being used on the work without additional tests in accordance, herewith, to show the new water-cement ratios to be used.

(b) During the progress of the work, a reasonable number of additional 28-day compression tests may be required by the Commissioner of Buildings, but at least one specimen shall be tested for each 50 cubic yards of concrete of any one strength, and not less than two specimens of each strength of concrete for any one day's operation. Such tests shall be made in accordance with the provisions of Section 304. Should the average strengths of the control cylinders shown by these tests for any portion of the structure fall below the minimum ultimate 28-day strengths called for on the plans, the Commissioner of Buildings shall have the right to order a change in the mix or the water-cement ratios for the remaining portion of the structure and to require load tests as specified in Section 202 on the portions of the building affected. Should the average strengths shown by the cylinders cured on the job and tested subsequent to 28 days fall below the required strength, the Commissioner of Buildings shall have the right to require conditions of temperature and moisture necessary to secure the required strength.

304: *Field Tests of Concrete:*

(a) Field tests of concrete, when required, shall be made in accordance with the "Standard Method of Making Compression Tests of Concrete" (Serial Designation C 39-27) of the American Society for Testing Materials with the following exceptions:

(1) Two sets of samples of concrete for test specimens shall be taken as the concrete is being delivered at the point of deposit, care being taken to obtain a sample representative of the entire batch.

(2) One set designated as control cylinders shall be placed under moist curing conditions at approximately 70 deg. F. within 24 hours after molding and maintained therein until tested.

(3) The second set, designated as job cylinders, shall be kept as near to the point of sampling as possible and yet receive the same protection from the elements as is given to the portions of the structure being placed. Specimens shall be kept from injury while on the work. They shall be sent to the laboratory not more than 7 days prior to the time of test and while in the laboratory shall be kept in the ordinary air at a temperature of approximately 70 deg. F.

(b) All specimens and tests shall be made by a properly qualified person or testing laboratory, who shall furnish the commissioner of buildings with a report, certified in the presence of a notary public, showing the results of tests and stating that they were made in accordance with the provisions of this code.

305: *Concrete Proportions and Consistency:*

(a) The proportions of aggregates to cement for concrete of any water-cement ratio shall be such as to produce concrete that will work readily into the corners and angles of the form and around the reinforcement without excessive puddling or spading and without permitting the materials to segregate or free water to collect on the surface. The combined aggregate shall be of such composition of sizes that when separated by the No. 4 standard sieve, the weight retained on the sieve shall not

be less than one-third nor more than two-thirds of the total nor shall the amount of coarse material be such as to produce harshness in placing or honeycombing in the structure. When forms are removed, the faces and corners of the members shall show smooth and sound throughout.

(b) The methods of measuring concrete materials shall be such that the proportion of water to cement can be accurately controlled during the progress of the work and easily checked at any time by the commissioner of buildings or his authorized representative.

306: *Allowable Unit Stresses in Concrete:*

(a) The unit stresses in pounds per square inch on the concrete to be used in the design shall not exceed the following values, where f'_c equals the minimum ultimate strength at 28 days.

DESCRIPTION	Allowable Unit Stresses: For any Strength of Concrete as Fixed by Test in Accordance with Sec. 303 $n = \frac{30000}{f'_c}$	When Strength of Concrete is Fixed by the Water-Cement Ratio in Accordance with Sec. 302: $f'_c =$ 2000 lb. $n = 15$	$f'_c =$ 2500 lb. $n = 12$	$f'_c =$ 3000 lb. $n = 10$
Flexure: fc.				
Extreme fiber stress in compression (*fc*)	$0.40f'_c$	800	1000	1200
Extreme fiber stress in compression adjacent to supports of continuous or fixed beams or of rigid frames (*fc*)	$0.45f'_c$	900	1125	1350
Shear: v.				
Beams with no web reinforcement and without special anchorage of longitudinal steel (*vc*)	$0.02f'_c$	40	50	60
Beams with no web reinforcement, but with special anchorage of longitudinal steel (*vc*)	$0.03f'_c$	60	75	90
Beams with properly designed web reinforcement, but without special anchorage of longitudinal steel (*v*)	$0.06f'_c$	120	150	180
Beams with properly designed web reinforcement and with special anchorage of longitudinal steel (*v*)	$0.09f'_c$	180	225	270
For conditions determining the use of greater shear values see Sec. 903(e).				
Flat slabs at distance d from edge of column cap or drop panel (*vc*)	$0.03f'_c$	60	75	90
Footings where longitudinal bars have no special anchorage (*vc*)	$0.02f'_c$	40	50	60
Footings where longitudinal bars have special anchorage (*vc*).	$0.03f'_c$	60	75	90
Bond: u.				
In beams and slabs and one-way footings:				
Plain bars (*u*)	$0.04f'_c$	80	100	120
Deformed bars (*u*)	$0.05f'_c$	100	125	150
In two-way footings:				
Plain bars (*u*)	$0.03f'_c$	60	75	90
Deformed bars (*u*)	$0.0375f'_c$	75	94	112
(Where special anchorage is provided (see Sec. 903), double these values in bond may be used.)				
Bearing: fc.				
Where a concrete member has an area at least twice the area in bearing (*fc*)	$0.25f'_c$	500	625	750
Axial Compression: fc.				
In columns with lateral ties (*fc*)	$0.225f'_c$	450	563	675
In columns with continuous spirals enclosing a circular core:[1]				
Ratio of longitudinal reinforcement p = 0.01	$300 + 0.14f'_c$	580	650	720
0.02	$300 + 0.18f'_c$	660	750	840
0.03	$300 + 0.22f'_c$	740	850	960
0.04	$300 + 0.26f'_c$	820	950	1080
0.05	$300 + 0.30f'_c$	900	1050	1200
0.06	$300 + 0.34f'_c$	980	1150	1320
(Spiral reinforcement not to be less than ¼ the longitudinal.)				

[1] Unit stress in spirally reinforced columns $= [300 + (0.10 + 4p)f'_c]$.

307: *Allowable Unit Stresses in Reinforcement:*

(a) The following unit stresses in reinforcing steel shall not be exceeded:

Tension:

Intermediate grade billet steel[1]............(f_s) = 20,000 lb. per sq. in.
Rail steel bars..........................(f_s) = 20,000 lb. per sq. in.
Web reinforcement ....(f_v) = 16,000 lb. per sq. in.
Structural steel shapes..................(f_s) = 18,000 lb. per sq. in.
Other steel reinforcement 50 per cent of the yield point stress, but not to exceed....(f_s) = 20,000 lb. per sq. in.

Compression:

Bars ....nf_c
Structural Steel section in composite columns......15,000 lb. per sq. in.
Cast Iron section in composite columns............ 9,000 lb. per sq. in.

See Section 1106 for stresses in structural steel and cast iron not encased in concrete.

Structural Steel section in combination column, see Section 1107.

CHAPTER 4.

MIXING AND PLACING CONCRETE.

401: *Removal of Water from Excavation:*

(a) Water shall be removed from excavations before concrete is deposited, unless otherwise directed by the commissioner of buildings. Any flow of water into the excavation shall be diverted through proper side drains to a sump, or be removed by other approved methods which will avoid washing the freshly deposited concrete. Water vent pipes and drains shall be filled by grouting or otherwise, after the concrete has thoroughly hardened.

402: *Cleaning Forms and Equipment:*

(a) Before placing concrete, all equipment for mixing and transporting the concrete shall be cleaned, all debris and ice shall be removed from the places to be occupied by the concrete, forms shall be thoroughly wetted (except in freezing weather) or oiled, and clay or cement tile that will be in contact with concrete shall be well drenched (except in freezing weather). Reinforcement shall be thoroughly cleaned of ice or other coatings.

403: *Inspection:*

(a) Concrete shall not be placed until the forms and reinforcement have been inspected by the architect or engineer responsible for the design or his authorized representative.

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[1] Until existing stocks of structural and hard grades of billet-steel reinforcement are exhausted, these grades, if conforming to the provision of Sec. 207, may be used with the following unit stresses:

Structural Grade ....(f_s) = 18,000 lb. per sq. in.
Hard Grade ....(f_s) = 20,000 lb. per sq. in.

404: *Mixing:*

(a) The concrete shall be mixed until there is a uniform distribution of the materials and the mass is uniform in color and homogeneous. The mixer shall be of such type as to insure the maintaining of the correct proportions of the ingredients. The mixing shall continue for at least one minute after all the ingredients are in the mixer.

405: *Transporting:*

(a) Concrete shall be handled from the mixer to the place of final deposit as rapidly as practicable by methods which will prevent the separation or loss of the ingredients. It shall be deposited as nearly as practicable in its final position to avoid rehandling or flowing. Under no circumstances shall concrete that has partially hardened be deposited in the work.

(b) When concrete is conveyed by chuting, the plant shall be of such size and design as to insure a practically continuous flow in the chute. The slope of the chute shall be such as to allow the concrete to flow without separation of the ingredients. The delivery end of the chute shall be as close as possible to the point of deposit. When the operation is intermittent, the spout shall discharge into a hopper. The chute shall be thoroughly flushed with water before and after each run; the water used for this purpose shall be discharged outside the forms.

406: *Placing:*

(a) When concreting is once started, it shall be carried on as a continuous operation until the placing of the section or panel is completed. Where construction joints are necessary, they shall be made in accordance with Sec. 507.

(b) Concrete shall be thoroughly compacted by puddling with suitable tools during the operation of placing, and thoroughly worked around the reinforcement, around embedded fixtures, and into the corners of the forms.

(c) Where conditions make puddling difficult, or where the reinforcement is congested, batches of mortar containing the same proportion of cement to sand used in the concrete, shall first be deposited in the forms and the operation of filling with the regularly specified mix be carried on at such a rate that the mix is at all times plastic and flows readily into the spaces between the bars.

(d) A record shall be kept on the work of the time and date of placing the concrete in each portion of the structure. Such record shall be kept until the completion of the structure and shall be open to the inspection of the commissioner of buildings.

407: *Curing:*

(a) Exposed surfaces of concrete shall be kept moist for a period of at least 7 days after being deposited. In hot weather, exposed concrete shall be thoroughly wetted twice daily during the first week.

408: *Depositing in Cold Weather:*

(a) When depositing concrete at freezing or near freezing temperatures, the concrete shall have a temperature of at least 50 deg. F., but not more than 120 deg. F. The concrete shall be maintained at a temperature of at least 50 deg. F. for not less than 72 hours after placing or until the concrete has thoroughly hardened. When necessary, concrete materials shall be heated before mixing. Dependence shall not be placed on salt or other chemicals for the prevention of freezing. No frozen materials or materials containing ice shall be used. Manure shall not be applied directly to concrete when used for protection.

CHAPTER 5.

FORMS AND DETAILS OF CONSTRUCTION.

501: *Design of Forms:*

(a) Forms shall conform to the shape, lines, and dimensions of the member as called for on the plans. They shall be substantial and sufficiently tight to prevent leakage of mortar; they shall be properly braced or tied together so as to maintain position and shape and insure safety to workmen and passersby. Temporary openings shall be provided where necessary, to facilitate cleaning and inspection immediately before depositing concrete.

502: *Removal of Forms:*

(a) The removal of forms shall be carried out in such a manner as to insure the complete safety of the structure. Where the structure as a whole is supported on shores, removable floor forms, beams and girder sides, column and similar vertical forms may be removed within 24 hours, providing the concrete has hardened sufficiently that it is not injured. In no case shall the supporting forms be disturbed until the concrete has hardened sufficiently to permit their removal with safety. Shoring shall not be removed until the member has acquired sufficient strength to support safely its weight and the load upon it.

503: *Cleaning and Bending Reinforcement:*

(a) Metal reinforcement, before being placed, shall be free from rust scale or other coatings that will destroy or reduce the bond. Reinforcement shall be formed to the dimensions indicated on the plans. Cold bends shall be made around a pin having a diameter of four or more times the least dimension of the bar.

(b) Metal reinforcement shall not be bent or straightened in a manner that will injure the material. Bars with kinks or bends not shown on the plans shall not be used. Heating of reinforcement for bending will not be permitted.

504: *Placing Reinforcement:*

(a) Metal reinforcement shall be accurately placed and secured, and shall be supported by concrete or metal chairs or spacers, or metal hangers. The minimum center to center distance between parallel bars shall be

2½ times the diameter for round bars or 3 times the side dimension for square bars; if the ends of bars are anchored as specified in Sec. 903, the center to center spacing may be made equal to 2 diameters for round bars or to 2½ times the side dimension for square bars, but in no case shall the clear spacing between bars be less than 1 in., nor less than 1⅓ times the maximum size of the coarse aggregate. Bars at the upper face of any member shall be embedded a clear distance of not less than one diameter, nor less than 1 in.

505: *Splices and Offsets in Reinforcement:*

(a) In slabs, beams, and girders, splices of reinforcement shall not be made at points of maximum stress without the approval of the commissioner of buildings. Splices, where permitted, shall provide sufficient lap to transfer the stress between bars by bond and shear. In such splices, the bars shall be spaced at the minimum distance specified in Sec. 504.

(b) Splices in column bars shall provide a lap of 24 diameters for deformed bars and 30 diameters for plain bars.

(c) Where changes in the cross-section of a column occur, the longitudinal bars shall be sloped for the full length of the column or offset in a region where lateral support is afforded. Where offset, the slope of the inclined portion from the axis of the column shall not be more than 1 in 6.

506: *Protective Covering of Concrete:*

(a) At those surfaces of footings and other principal structural members in which the concrete is deposited directly against the ground, metal reinforcement shall have a minimum covering of 3 in. of concrete. At other surfaces of concrete exposed to the ground or weather, metal reinforcement shall be protected by not less than 2 in. of concrete.

(b) In fire-resistive construction, metal reinforcement shall be protected by not less than 1 in. of concrete in slabs and walls, and not less than 1½ in. in beams, girders, and columns, provided coarse aggregate is used, which is free from disruptive action under high temperatures, as, for example, limestone or trap rock; when impracticable to obtain aggregate of this grade, the protective covering shall be ½ in. thicker and shall be reinforced with metal mesh having openings not exceeding 3 in. placed 1 in. from the finished surface. In similar structures where the fire hazard is limited, the metal reinforcement shall not be placed nearer the exposed surface than ¾ in. in slabs and walls, or 1 in. in beams, girders, and columns.

(c) Cement or gypsum plaster, ¾ in. or more in thickness (on metal lath weighing not less than 2½ lb. per sq. yd. when used vertically, nor less than 3 lb. per sq. yd. when used horizontally) may be substituted for a part of the protective covering of concrete, provided that only two-thirds of the thickness of the plaster be considered effective and the concrete protection shall in no case be reduced to less than ¾ in.

(d) Exposed reinforcement bars intended for bonding with future extensions shall be protected from corrosion.

507: *Construction Joints:*

(a) Joints not indicated on the plans shall be so made and located as to least impair the strength of the structure. Where a horizontal joint is to be made, any excess water and laitance shall be removed from the surface after concrete is deposited. Before depositing of concrete is resumed, the hardened surface shall be cleaned and roughened and all weak concrete removed.

(b) At least 2 hours must elapse after depositing concrete in the columns or walls before depositing in beams, girders, or slabs supported thereon. Beams, girders, brackets, column capitals, and haunches shall be considered as part of the floor system and shall be placed monolithically therewith.

(c) Construction joints in floors shall be located near the middle of spans of slabs, beams, or girders, unless a beam intersects a girder at this point, in which case the joints in the girders shall be offset a distance equal to twice the width of the beam. In this last case provision shall be made for shear by use of inclined reinforcement.

CHAPTER 6.

DESIGN—GENERAL CONSIDERATIONS.

601: *Assumptions:*

(a) The design of reinforced-concrete members under these specifications shall be made with reference to working stresses and safe loads. The accepted theory of flexure as applied to reinforced concrete shall be applied to all members resisting bending involving the following assumptions:

(1) The steel takes all tensile stress,

(2) The ratio n of the modulus of elasticity of the steel to that of the concrete shall be taken as follows (applies also for compression members):

$$n = \frac{E_s}{1{,}000f'_c} = \frac{30{,}000}{f'_c}$$

602: *Notation:*

(a) The symbols and notation used in these regulations are defined as follows:

a = width of face of column or pedestal;

α = angle between inclined web bars and axis of beam;

A = total area of top of pedestal, pier, or footing;

A' = loaded area of pedestal, pier, or footing at the column base;

A_c = area of core of spirally-hooped column measured to the outside diameter of the spiral;

A_g = gross area of tied columns with lateral ties;

A_s = effective cross-sectional area of metal reinforcement in tension in beams or compression in columns; and the effective cross-sectional area of metal reinforcement which crosses any of the principal design sections of a flat slab and which meets the requirements of Sec. 1006, 1008, 1009, and 1010.

A_v = total area of web reinforcement in tension within a distance of s (measured perpendicular to the direction of the web reinforcement bar), or the total area of all bars bent up in any one plane;

b = width of rectangular beam or width of flange of T-beam;

b' = thickness of web in beams of I or T sections;

b_1 = dimension of the dropped panel of a flat slab in the direction parallel to l_1;

c = diameter in feet of column capital of a flat slab at the underside of the slab, or dropped panel. No portion of the column capital shall be considered for structural purposes which lies outside of the largest 90° cone that can be included within the outlines of the column capital;

c = projection of footing from face of column or pedestal;

d = depth from compression surface of beam or slab to center of longitudinal tensile reinforcement;

E_c = modulus of elasticity of concrete in compression;

E_s = modulus of elasticity of steel in tension or compression = 30,000,000 lb. per sq. in.;

f_c = compressive unit stress in extreme fiber of concrete in flexure or axial compression in concrete in columns;

f'_c = ultimate compressive strength of concrete at age of 28 days;

f_r = compressive unit stress in metal core;

f_s = tensile unit stress in longitudinal reinforcement;

f_v = tensile unit stress in web reinforcement;

h = unsupported length of column;

I = moment of inertia of a section about the neutral axis for bending;

l = span length of beam or slab (generally distance from center to center of supports; for special cases, see Sec. 702 and 1005);

l = span length of flat slab panel (usually expressed in feet) center to center of columns, in the direction in which moments are considered (see Sec. 1003);

l_1 = span length of flat slab, center to center of columns, perpendicular to the rectangular direction in which moments are considered;

M = bending moment or moment of resistance in general;

M_o = sum of positive and negative bending moments at the principal design sections of a panel of a flat slab (see Sec. 1003);

n = E_s/E_c = ratio of modulus of elasticity of steel to that of concrete;

Σo = sum of perimeters of bars in one set;

p = ratio of effective area of tensile reinforcement to effective area of concrete in beams = A_s/bd; and the ratio of effective area of longitudinal reinforcement to the area of the concrete core in columns;

p_a = permissible unit stress on pedestal, pier, or footing when the full area is loaded;
P = total safe axial load on column whose length does not exceed 11 times its least cross-sectional dimension;
P' = total safe axial load on long column;
r_a = permissible unit working stress in concrete over the loaded area of a pedestal, pier, or footing;
R = least radius of gyration of a section;
s = spacing of stirrups measured perpendicular to the direction of the stirrup;
t = thickness of flange of T-beam;
t_1 = thickness of flat slab without dropped panels; or the thickness of flat slabs, including dropped panels where one is used;
t_2 = thickness of flat slab with dropped panels at points away from the dropped panel;
u = bond stress per unit of area of surface of bar;
v = shearing unit stress;
v_c = unit shearing stress permitted on the concrete of the web; the value depending on the anchorage of the longitudinal reinforcement;
V = total shear;
V' = excess of the total shear over that permitted on the concrete;
w = uniformly distributed load per unit of length of beam or slab;
w = upward reaction per unit of area of base of footing;
w' = uniformly distributed dead and live load per unit of area of a floor or roof (in flat slabs usually expressed in pounds per square foot);
W = total dead and live load uniformly distributed over a single panel area (in flat slabs usually expressed in pounds and includes the dead weight of any raised or depressed portions).

603: *Design Loads:*

(a) The provisions for design herein specified are based on the assumption that all structures shall be designed for all dead- and live-loads coming upon them, the live-loads to be in accordance with the general requirements of the building code of which this forms a part, with such reductions for girders and lower story columns as are permitted therein.

604: *Wind Loads:*

(a) Provisions shall be made for wind loads in accordance with the general provisions of the code of which this forms a part. In designing the members to resist wind loads, the allowable unit stresses for dead- and live-load and wind loads may be increased to 150 per cent of the allowable values specified in Sec. 306 and 307, but the section shall not be less than that required if the wind load be neglected.

CHAPTER 7.

FLEXURAL COMPUTATIONS AND MOMENT COEFFICIENTS.

701: *Formulas for Flexure:*

(a) Computations of flexural resistance of reinforced-concrete beams and slabs shall be based on the assumptions of Sec. 601. The customary formulas or their equivalent shall be used.

702: *Span Length:*

(a) The span length of freely supported beams and slabs shall be the clear span plus the depth of beam or slab, but shall not exceed the distance between centers of the supports.

(b) The span length for continuous or restrained beams built to act integrally with supports shall be the clear distance between faces of supports.

(c) For continuous or restrained beams having brackets built to act integrally with both beam and support and of a width not less than the width of the beam and making an angle of 45 deg. or more with the horizontal, the span shall be measured from the section where the combined depth of the beam and bracket is at least one-third more than the depth of the beam. No portion of such a bracket shall be considered as adding to the effective depth of the beam. Brackets making an angle of less than 45 deg. with the horizontal may be considered as increasing the effective depth of the beam, but not as decreasing the span length.

(d) Maximum negative moments are to be considered as existing at the ends of the span, as defined above.

703: *Depth of Beam or Slab:*

(a) The depth of the beam or slab shall be taken as the distance from the centroid of the tensile reinforcement to the top surface of the structural slab. Any floor finish not placed monolithic with the floor slab shall not be included as a part of the structural member. When the finish is placed monolithic with the structural slab in buildings of the warehouse or industrial class where the finish is subjected to unusual wear from trucking or other causes, there shall be placed an additional depth of ½ in. over that used in the design of the member.

704: *Point of Inflection:*

(a) For the purpose of these regulations, the point of inflection in beams and slabs of equal spans symmetrically loaded shall be assumed to be located at the fifth point of the span as defined in Sec. 702.

705: *Distance between Lateral Supports:*

(a) The clear distance between lateral supports of a beam shall not exceed 32 times the least width of compression flange.

706: *Requirements for T-Beams:*

(a) In T-beam construction the slab shall be built integrally with the beam. The effective flange width to be used in the design of symmetrical T-beams shall not exceed one-fourth of the span length of the beam,

and its overhanging width on either side of the web shall not exceed eight times the thickness of the slab nor one-half the clear distance to the next beam.

(b) For beams having a flange on one side only, the effective overhanging flange width shall not exceed one-twelfth of the span length of the beam, nor six times the thickness of the slab, nor one-half the clear distance to the next beam.

(c) Where the principal reinforcement in a slab which is considered as the flange of a T-beam (not a rib in ribbed floors) is parallel to the beam, transverse reinforcement shall be provided in the top of the slab. This reinforcement shall be designed to carry the load on the portion of the slab assumed as the flange of the T-beam. The spacing of the bars shall not exceed five times the thickness of the flange, or in any case 18 in.

(d) Provision shall be made for the compressive stress at the support in continuous T-beam construction, care being taken that the provisions of Sec. 504, relating to the spacing of bars, and 406(c), relating to the placing of concrete shall be fully met. In no case shall the area of steel in compression at any cross-section adjacent to the support exceed 2 per cent of the cross-sectional area of the stem of the beam in that section.

(e) The overhanging portion of the flange of the beam shall not be considered as effective in computing the shear and diagonal tension resistance of T-beams.

(f) Isolated beams in which the T-form is used only for purpose of providing additional compression area, shall have a flange thickness not less than one-half the width of the web and a total flange width not more than four times the web thickness.

707: *Ribbed Floor Construction:*

(a) Ribbed floor construction includes floor systems of ribs and slabs placed monolithically in which the ribs are not farther apart than 36 in. face to face. The ribs shall be straight, not less than 4 in. wide, nor of a depth more than 3 times the width.

(b) Where removable forms or fillers not complying with (c) are used the thickness of the concrete slab shall not be less than 1/12 of the clear distance between ribs and in no case less than 2 in.

(c) When burned clay or cement tile are used and concrete is placed on the top of such tile, it shall not be less than 1½ in. in thickness, nor less than one-twelfth of the clear distance between ribs. When the tile are so placed that the joints in alternate rows are staggered, the webs of the tile in contact with the ribs may be included in calculations involving shear or negative bending moment. No other portion of the tile may be included in design calculations.

(d) Where the floor is subject to impact from moving loads, or to wear, the slab thicknesses shall be increased ½ in. If a floor or covering ½ in. or more in thickness, not included in the structural slab, is used for a wearing surface, no increase need be made.

(e) Where the slab contains conduits or pipes, the thickness shall not be less than 1 in. plus the total overall depth of such conduits or pipes at any point. Such conduits or pipes shall be so located as not to reduce the strength of the construction.

(f) Shrinkage reinforcement in the slab must be provided as required in Section 712.

708: *Moment Coefficients for Freely Supported or Slightly Restrained Continuous Beams or Slabs of Approximately Equal Span; Uniform Load:*

(a) Beams and slabs of approximately equal spans freely supported or built to act integrally with beams, girders, or other slightly restraining support, or beams and slabs built into brick or masonry walls in a manner which develops only partial end restraint, and carrying uniformly distributed loads shall be designed for the following moments at critical sections:

(1) Beams and slabs of one span,
Positive moment near center,

$$M = \frac{wl^2}{8} \quad \ldots\ldots\ldots\ldots \quad (1)$$

(2) Beams and slabs continuous for two spans only,
Positive moment near center,

$$M = \frac{wl^2}{10} \quad \ldots\ldots\ldots\ldots \quad (2)$$

Negative moment over interior support,

$$M = \frac{wl^2}{8} \quad \ldots\ldots\ldots\ldots \quad (3)$$

(3) Beams and slabs continuous for more than two spans,
Positive moment near center and negative moment at support of interior spans,

$$M = \frac{wl^2}{12} \quad \ldots\ldots\ldots\ldots \quad (4)$$

Positive moment near centers of end spans and negative moment at first interior support,

$$M = \frac{wl^2}{10} \quad \ldots\ldots\ldots\ldots \quad (5)$$

(4) Negative moment at end supports for cases (1), (2), and (3) of this section,

$$M = \text{not less than } \frac{wl^2}{24} \quad \ldots\ldots\ldots\ldots \quad (6)$$

709: *Moment Coefficients for Fully Restrained Continuous Beams or Slabs of Approximately Equal Span: Uniform Load:*

(a) Beams and slabs of approximately equal spans built to act integrally with columns, walls, or other restraining supports and assumed to carry uniformly distributed loads, shall (except as provided in Sec. 708) be designed for the following moments at critical sections:

(1) Interior spans;

Negative moment at interior supports except the first,

$$M=\frac{wl^2}{12} \quad \ldots\ldots (7)$$

Positive moment near centers of interior spans,

$$M=\frac{wl^2}{16} \quad \ldots\ldots (8)$$

(2) End spans of continuous beams or slabs, and beams or slabs of one span;

Where I/l is less than twice the sum of the values of I/h for the exterior columns above and below which are built into the beams:

Positive moment near center of span and negative moment at first interior supports,

$$M=\frac{wl^2}{12} \quad \ldots\ldots (9)$$

Negative moment at exterior supports,

$$M=\frac{wl^2}{12} \quad \ldots\ldots (10)$$

Where I/l is equal to or greater than twice the sum of the values of I/h for the exterior column above and below which are built into the beams:

Positive moment near center of span and negative moment at first interior support,

$$M=\frac{wl^2}{10} \quad \ldots\ldots (11)$$

Negative moment at exterior support,

$$M=\frac{wl^2}{16} \quad \ldots\ldots (12)$$

(b) In this section, I represents the moment of inertia which, for those calculations, shall be computed on the assumption that the member is homogeneous, neglecting the reinforcement, but including that portion of the concrete section outside of the reinforcement which is ordinarily con-

sidered as fireproofing. l and h are the span length and column height, respectively, as defined in Sec. 702 and 1102.

710: *Moment Coefficients for Continuous Beams or Slabs of Unequal Span or with Non-Uniform Loads:*

(a) Continuous beams with substantially unequal spans, or with other than uniformly distributed loading, whether freely supported or restrained, shall be designed for the maximum moments resulting from the most severe probable combination of loading and restraint. Provision shall be made where necessary for negative moment near the center of short spans which are adjacent to long spans, and for the negative moment at the end supports, if restrained.

711: *Compression Steel in Flexural Members:*

(a) Where it is necessary to introduce steel in compression in girders, beams, or slabs, such steel shall be thoroughly anchored by ties or stirrups not less than ¼ in. in size which shall be spaced not more than 8 in. apart over the distance where the compression steel is required.

712: *Shrinkage and Temperature Reinforcement:*

(a) Reinforcement for shrinkage and temperature stresses normal to the principal reinforcement shall be provided in floor and roof slabs where the principal reinforcement extends in one direction only. Such reinforcement shall provide for the following minimum ratios of reinforcement area to concrete area, but in no case shall such reinforcing bars be placed farther apart than five times the slab thickness nor more than 18 in.:

Floor slabs where plain bars are used	0.0025
Floor slabs where deformed bars are used	0.002
Floor slabs where wire fabric is used, having welded intersections not farther apart in the direction of stress than 12 in.	0.0018
Roof slabs where plain bars are used	0.003
Roof slabs where deformed bars are used	0.0025
Roof slabs where wire fabric is used, having welded intersections not farther apart in the direction of stress than 12 in.	0.0022

*713: *Floors Reinforced in Two Directions:*

(a) Concrete floors supported on four sides by beams, girders, or walls, and reinforced in two directions, shall be designed as follows, using moment coefficients given in Section 708, 709, and 710 as required, except as indicated under (e).

(b) If the length of the slab exceeds one and one-half times its width, the entire load shall be carried in the short direction.

(c) In case of square panels and uniformly distributed load, one-half the live- and dead-load may be assumed as being resisted by each cross band.

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* The committee feels that this section may be too conservative. However, the additional investigation necessary to determine proper design methods requires more time than has been available.

(d) In rectangular panels of length L and breadth B, the portion of the load which shall be assumed as being supported by the slab in the short direction shall be equal to $\left(\frac{L}{B} - \frac{1}{2}\right)$ times the total load. The remainder of the load shall be assumed as being supported by the slab in the long direction. The reinforcement in the long direction shall in no case be less than that specified in Sec. 712 for shrinkage and temperature reinforcement.

(e) In placing the reinforcement account may be taken of the facts that the moment is less in the portions of the band which are adjacent and parallel to the supporting beams. In the one-quarter width of band parallel and adjacent to the beams, the computed moment may be reduced 50 per cent.

(f) Beams supporting such slabs shall be assumed to take the portion of the load as determined in (b), (c), or (d) without advantage of any reduction in live-load permitted in other sections of this code. The total load for each beam shall be assumed as uniformly distributed.

714: *Maximum Spacing of Principal Slab Reinforcement:*

(a) In slabs other than ribbed floor construction or flat slabs, the principal reinforcement shall not be spaced farther apart than three times the slab thickness, nor shall the ratio of reinforcement be less than specified in Section 712 (a).

CHAPTER 8.

SHEAR AND DIAGONAL TENSION.

801: *Shearing Unit Stress:*

(a) The shearing unit stress (v) in reinforced-concrete beams shall be computed by formula (14):

$$v = \frac{8V}{7\,bd} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (14)$$

When the value of the shearing unit stress computed by formula (14) exceeds the unit shearing stress (v_c) permitted on the concrete of the web (see 306-a), web reinforcement shall be provided to carry the excess.

(b) For beams of I or T section b' shall be substituted for b in formula (14).

(c) In tile and joist construction, b may be taken as a width equal to the thickness of the concrete web plus the thickness of the vertical webs of the concrete or clay tile in contact with the joist as in Sec. 707c.

802: *Types of Web Reinforcement:*

(a) Web reinforcement may consist of:

(1) Vertical stirrups or web reinforcing bars;

(2) Inclined stirrups or web reinforcing bars forming an angle of 30 deg. or more with the axis of the beam.

(3) Longitudinal bars bent up at an angle of 15 deg. or more with the axis of the beam.

(b) Stirrups or bent-up bars to be considered effective as web reinforcement shall be anchored at both ends, according to the provisions of Sec. 904.

803: *Stirrups:*

(a) Area of steel required in stirrup shall be computed by formula (15).

$$A_v = \frac{V's}{14000d} \quad \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (15)$$

804: *Spacing of Stirrups:*

(a) Where the shearing stress is not greater than $0.06f'_c$ the distance s between two successive stirrups measured perpendicular to the direction of the stirrup shall not exceed $\frac{3}{4}d$, and where unit shearing stress exceeds $0.06f'_c$, it shall not be greater than $\frac{3}{8}d$.

805: *Bent-up Bars:*

(a) Where there is a series of parallel bent-up bars at varying distances from the support, they shall be considered as inclined stirrups and the area required determined from formula (15).

(b) Where bent-up bars in a single plane are used for web reinforcement, the required area of the bar shall be computed by formula (16).

$$A_v = \frac{V'}{16000 \sin \alpha} \quad \dots\dots\dots\dots\dots\dots\dots\dots (16)$$

(c) In formula (16), V' shall not exceed $0.035f'_c\, bd$ nor α be less than 15 deg. Only the center three-fourths of the inclined portion of such bar or group of bars shall be considered effective in resisting shear. Between the face of the support and the area reinforced by the bent-up bar, other web reinforcement shall be provided, except that when the distance is less than $d/2$ and the beam is designed for uniform load only, such additional reinforcement need not be provided.

806: *Combined Web Reinforcement:*

(a) Where two or more types of web reinforcement are used in conjunction, the total shearing resistance of the beam shall be assumed as the sum of the shearing resistances computed for the various types separately. In such computations the shearing resistance of the concrete shall be included only once.

807: *Shearing Stress in Flat Slabs:*

(a) In flat slabs, the shearing unit stress on a vertical section which lies at a distance $t_1 - 1\frac{1}{2}$ in. from the edge of the column capital and parallel with it, shall not exceed the following values when computed by formula (14) (in which d shall be taken as $t_1 - 1\frac{1}{2}$ in.):

(1) $0.03f'_c$ when at least 50 per cent of the total negative reinforcement passes directly over the column capital;

(2) $0.025f'_c$ when 25 per cent of the total negative reinforce-

ment passes directly over the column capital (which is the least that shall be permitted);

(3) For intermediate percentages, intermediate values of the shearing unit stress shall be used.

(b) In flat slabs, the shearing unit stress on a vertical section which lies at a distance of $t_2 - 1\frac{1}{2}$ in. from the edge of the dropped panel and parallel with it shall not exceed $0.03f'_c$ when computed by formula (14) (in which d shall be taken as $t_2 - 1\frac{1}{2}$ in.). At least 50 per cent of the cross-sectional area of the negative reinforcement in two column strips must be within the width of strip directly above the dropped panel.

808: *Shear and Diagonal Tension in Footings:*

(a) The shearing unit stress computed by formula (14) on a vertical section, which lies at a distance d from the face of the supported column or pier and parallel with it, shall not exceed $0.02f'_c$ for footings with straight bars, nor $0.03f'_c$ for footings in which the bars are anchored at both ends by adequate hooks or otherwise specified in Sec. 903.

(b) In footings supported on piles, the critical section for diagonal tension shall be considered the distance $d/2$ from the face of the column or pedestal and any piles whose centers are at or within this section shall be excluded in computing the shear.

CHAPTER 9.

BOND AND ANCHORAGE

901: *Computation of Bond Stress in Beams:*

(a) Where reinforcement is used to resist tensile stresses developed by beam action, the bond stress shall be taken as not less than that computed by formula (17).

$$u = \frac{8V}{7\,\Sigma od} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

(b) For continuous or restrained members, the critical section for bond for the positive reinforcement shall be assumed to be at the point of inflection, that for the negative reinforcement shall be assumed to be at the face of the support, and at the point of inflection. For simple beams, or at the outer ends of freely supported end spans of continuous beams, the critical section for bond shall be assumed to be at the face of the support.

(c) Bent-up longitudinal bars which, at the critical section, are within a distance $d/3$ from horizontal reinforcement under consideration may be included with the straight bars in computing Σo.

902: *Ordinary Anchorage Requirements:*

(a) Tensile negative reinforcement in any span of a continuous, restrained, or cantilever beam, or in any member of a rigid frame, shall have a length of anchorage beyond the face of the supporting member sufficient to develop the full maximum tension at an average bond stress not greater

than $0.04f'_c$, for plain bars, or $0.05f'_c$ for deformed bars. Within any such span, not less than one-third of the negative reinforcements shall extend along the tension side of the beam at least to or beyond the point of inflection, and any bars not so extended shall be bent down at an angle of not more than 45 deg. with the axis of the member and made continuous with the positive reinforcement or anchored in a region of compression.

(b) Of the positive reinforcement in continuous beams, not less than one-fourth the area shall extend at the same face of the beam into the support to provide an embedment of ten or more bar diameters beyond the face of the support.

(c) For non-continuous beams not less than one-half the area of positive reinforcement shall extend at the same face of the beam into the support to provide an embedment of ten or more bar diameters.

903: *Special Anchorage Requirements:*

(a) Where increased shearing or bond stresses on account of special anchorages are permitted as specified in Section 306, anchorage of all reinforcement as required in Section 902 shall be increased to conform with the requirements of (b), (c), (d), and (e) of this section.

(b) In continuous and restrained beams, anchorage beyond points of inflection of at least one-third the area of the negative reinforcement and beyond the face of the support of at least one-third the area of the positive reinforcement, shall be provided to develop one-third of the allowable working stress in tension at average bond stresses not to exceed $0.04f'_c$ for plain bars nor $0.05f'_c$ for deformed bars.

(c) In footings, all bars shall be anchored by means of hooks at the end of the bar. The total length of bar shall be the width of the footing plus 20 bar diameters. The outer face of the hook shall not be less than 3 in. nor more than 4 in. from the face of the footing.

(d) In simple beams, or at the outer ends of freely supported end spans of continuous beams, at least one-half of the tensile reinforcement shall extend along the tension side of the beam to provide an anchorage beyond the face of the support for one-third of the allowable working stress in tension at an average bond stress not to exceed $0.04f'_c$ for plain bars, nor $0.05f'_c$ for deformed bars.

(e) In cases where the design of unusual members involves the use of unit shearing stresses in excess of $0.09f'_c$, values up to $0.12f'_c$ may be used, providing the requirements of this section are fully met, that the members in which these stresses are used shall be specially designated on the plans and that these members shall be constructed under the personal supervision of the designing engineer who shall notify the commissioner of buildings at least one day in advance of the placing of the concrete in such member. When required by the commissioner of buildings, the designing engineer shall submit an affidavit certifying that he has personally supervised the construction of these members and that the design and construction was in all respects as called for on the plans and in conformity with the provisions of this code.

904: *Anchorage of Web Reinforcement:*

(a) Web reinforcement shall be anchored at both ends by one of the following methods or combination thereof, but only anchorage meeting the requirements of (1), (2) or (3) shall be used for shearing unit stresses in excess of $0.08f'_c$.

(1) Providing continuity with the main longitudinal reinforcement.

(2) Bending around the longitudinal bar or steel shape;

(3) A hook which has a radius of bend not less than 4 times the diameter of the web bar;

(4) A length of embedment sufficient to develop the stress in the stirrup by bond as provided below, provided also that the other end of the stirrup is anchored as in (1).

(b) The end anchorage of a web member not bearing on the longitudinal reinforcement shall be such as to engage an amount of concrete sufficient to prevent the bar from pulling out. In all cases the stirrups shall be carried as close to the upper and lower surfaces as fireproofing requirements permit.

(c) The stress in a stirrup or web reinforcement bar shall not exceed a value equal to the surface area of the bar embedded within the upper or lower one-half of the beam multiplied by $0.04f'_c$ for plain bars, or $0.05f'_c$ for deformed bars, except that when wire fabric is used for web reinforcement it shall have welded intersections not farther apart than 6 in., but in no case shall the stress exceed 16,000 lb. per sq. in.

CHAPTER 10.

FLAT SLABS.

(Two-Way and Four-Way Systems with Square or Rectangular Panels.)

1001: *Limitations:*

(a) The term flat slabs as used in these regulations refers to concrete slabs, having reinforcement bars extending in two or four directions, without beams or girders to carry the load to supporting members.

(b) The moment coefficients, moment distribution, and slab thicknesses specified herein are for a series of slabs of approximately uniform size arranged in three or more rows of panels in each direction, and in which the ratio of length to width of panel does not exceed 1.33.

(c) Slabs with paneled ceiling or with dropped panels shall be considered as coming under the requirements herein given, provided the dropped panel shall have a length or diameter in each direction parallel to a side of the panel of not less than 0.35 of the panel length in that direction, and provided further that the depth of the thicker portion of the slab does not exceed one and one-half times the depth of the remainder of the slab.

(d) For structures having a width of less than three rows of panels, or in which irregular panels are used, an analysis shall be made of the moments developed in both slabs and columns. When so required, computations shall be submitted to the commissioner of buildings for approval.

1002: *Panel Strips and Principal Design Sections:*

(a) For convenience of reference, a flat slab panel shall be considered as consisting of strips as follows:

A *middle strip* one-half panel in width symmetrical with respect to the panel center line and extending through the panel in the direction in which moments are being considered;

A *column strip* one-half panel in width occupying the two quarter panel areas outside of the middle strip.

When considering moments in the direction of the width of the panel, the panel is similarly divided by strips, the widths of which are each one-half the length of the panel.

(b) The critical sections for moment calculations are referred to as principal design sections and are located as follows:

Sections for Negative Moment. These shall be taken along the edges of the panel, on lines joining the column centers, and following the circumference of the column capital.

Sections for Positive Moment. These shall be taken on the center line of the panel.

1003: *Moments in Interior Panels—General Case:*

(a) The numerical sum of the positive and negative moments in the direction of either side of a rectangular panel shall be not less than that given by formula (19).

$$M_0 = 0.09Wl\left(1 - \frac{2c}{3l}\right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (19)$$

where M_0 = sum of positive and negative bending moments at the principal design sections, in the direction in which the length is given by l. This moment is in foot-pounds when c and l are in feet and W is in pounds.

(b) The moments in the principal design sections shall be those given in the following table of moments, except that the maximum negative moment in the column strip may be greater or less than the values given in the table of moments by not more than $0.03M_0$, provided that the sum of the moments on the principal section remains equal to M_0, and provided further that the moment in each of the three other critical design sections be modified by not more than $0.01M_0$.

MOMENTS TO BE USED IN DESIGN OF FLAT SLABS

For Interior Panels Fully Continuous

General case: all values of c: M_o given by formula (19)

Strip	Flat Slabs without Dropped Panels		Flat Slabs with Dropped Panels	
	Negative	Positive	Negative	Positive
Slabs with 2-Way Reinforcement				
Column Strip..........	$-M_c = 0.46M_o$	$+M_c = 0.22M_o$	$-M_c = 0.50M_o$	$+M_c = 0.20M_o$
Middle Strip...........	$-M_m = 0.16M_o$	$+M_m = 0.16M_o$	$-M_m = 0.15M_o$	$+M_m = 0.15M_o$
Slabs with 4-Way Reinforcement				
Column Strip.........	$-M_c = 0.50M_o$	$+M_c = 0.20M_o$	$-M_c = 0.54M_o$	$+M_c = 0.19M_o$
Middle Strip...........	$-M_m = 0.10M_o$	$+M_m = 0.20M_o$	$-M_m = 0.08M_o$	$+M_m = 0.19M_o$

(c) The width of section at the column head shall be taken as the width of the dropped panel where used or half the width of panel where no dropped panel is used.

(d) The band width in the two-way system shall be such as to provide reinforcement over the entire one-half panel width.

(e) The band width for the direct bands in the 4-way system shall be approximately 4/10 of the panel width at right angles to the direction of the band ($0.4l_1$) and for the diagonal bands approximately 0.4 of the average span length. In proportioning the reinforcement in this system, it shall be assumed that reinforcement in the direct band resists the entire positive moment for the column strip and the two diagonal bands resist the entire positive moment for the middle strip. Reinforcement for negative moment for the column strip shall include the area of reinforcement for negative moment in the diagonal bands multiplied by the cosine of the angle between the diagonal band and the axis of the direct band considered plus the full area of the reinforcement for negative moment in the direct band. The negative reinforcement for the middle strip shall be provided independently of the diagonal bands.

1004: *Moments in Interior Panels — Special Case, $c = 0.225$ times the average span length:*

(a) For the particular case where c is equal to 0.225 times the average span length (the average of the distances center to center of columns on the two sides of the panel), formula (19) reduces to formula (19a).

$$M_o = 0.065Wl \quad \text{(19a)}$$

(b) For two-way slab, the values of M_o may be obtained from formula (19a) and the distribution taken from the table in Sec. 1003(b).

(c) For the four-way slab with dropped panel, the following table of coefficients may be used in computing the reinforcement required in each of the bands, provided that l for the direct bands shall be the center

to center distance between columns in the direction in which the band extends, and for the diagonal bands the average value of l for the two direct bands of the panel. The moments in the table are those on *sections at right angles* to the direction of the respective bands:

BAND	LOCATION	AMOUNT
Direct	Center	+0.012Wl
Diagonal	Center	+0.009Wl
Direct	At column head	—0.020Wl
Diagonal	At column head	—0.011Wl
Top band across direct band....	Between columns	—0.005Wl

1005: *Thickness of Slabs and Dropped Panels:*

(a) For slabs without dropped panels, using concrete of 2,000 lb. per sq. in. ultimate strength, the total thickness of the slab t_1, in inches, shall be not less than the value given by formula (20).

$$t_1 = 0.038\ (1 - 1.44\frac{c}{l})\,l\,\sqrt{w'} + 1\tfrac{1}{2} \quad \ldots\ldots\ldots\ldots (20)$$

where w' = uniformly distributed dead and live-load, lb. per sq. ft.

(b) For slabs with dropped panels, using concrete of 2,000 lb. per sq. in. ultimate strength, the total thickness in inches at points beyond the dropped panel shall be not less than

$$t_2 = 0.02l\,\sqrt{w'} + 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

(c) The dropped panel shall have a thickness not greater than $1.5t_2$ nor less than $1.25t_2$. The side or diameter of the dropped panel shall not be less than 0.35 times the side of the panel in the parallel direction.

(d) In determining minimum thickness by formulas (20) and (21), the value of l shall be the panel length center to center of the columns, on the long side of the panel. For concrete of 2,000 lb. per sq. in. ultimate strength, the slab thickness t_1 or t_2 shall in no case be less than $l/32$ for floor slabs, and not less than $l/40$ for roof slabs.

(e) Where concretes of higher ultimate strengths than 2,000 lb. per sq. in. are used, the thickness given by formulas (20) and (21) and the limiting thicknesses may be reduced by multiplying by the factor $\sqrt[3]{\frac{2{,}000}{f'_c}}$, in which f'_c is the ultimate strength of the concrete to be used.

1006: *Limiting Percentages of Reinforcement:*

(a) The ratio of reinforcement for negative moment in the column strip shall not exceed the values of p calculated for balanced reinforcement, that is, the amount of reinforcement for which both the steel and the concrete are stressed to the full amount permitted by Sec. 306 and 307. Any reinforcement in excess of this amount shall not be included in the calculation. In computing the ratio of reinforcement for negative moment in the column strip, the width of section shall be taken as in Sec. 1003(c).

In the case of four-way design, the steel area shall consist of the area of steel for negative moment as defined in 1003(e).

(b) The ratio of flat slab reinforcement in any strip shall not be less than .0025. Bars shall not be spaced farther apart than 1½ times the slab thickness.

1007: *Point of Inflection:*

(a) In the middle strip the point of inflection for slabs without dropped panels shall be assumed at a line $0.33l$ distant from the center of the span and for slabs with dropped panels $0.3l$ distant from the center of the span.

(b) In the column strip, the point of inflection for slabs without dropped panels shall be at a line $0.33\ (l - c)$ distant from the center of the panel and $0.3\ (l - c)$ for slabs with dropped panels.

1008: *Arrangement of Reinforcement at Column Heads—Two- and Four-Way Systems:*

(a) In both two- and four-way systems, provision shall be made for securing the reinforcement in place so as to resist properly not only the critical moments, but also the moments at intermediate sections. The full area of steel required for negative moment at the column head shall be continued in the same plane close to the upper surface of the slab to the edge of the dropped panel, but in no case less than a distance $0.2l$ from the center line of column. Lapped splices shall not be permitted at or near regions of maximum stress except as described in Sec. 505.

1009: *Arrangement of Reinforcement—Two-Way System:*

(a) For column strips at least four-tenths of the area of steel required at the section for positive moment in the column strip shall be of such length and so placed as to reinforce the negative moment section at the two adjacent column heads. These bars, and any other bars for negative reinforcement shall extend into the adjacent panel to a point at least $0.05l$ beyond the point of inflection. Not less than one-third of the bars used for positive reinforcement in the column strip shall extend into the dropped panel at least twenty diameters of the bar, but not less than 12 in. or in case no dropped panel is used, shall extend to within $0.125l$ of the center line of the columns or the supports. The balance of the bars for positive reinforcement in the column strip shall extend at least $0.33l$ on either side of the center line of panel.

(b) For the middle strip at least one-half of the bars for positive moment shall be bent up and extend over the main bands at both sides of the panel to a point at least $0.25l$ beyond the center line of columns. The location of the bends shall be such that for a distance $0.15l$ for slabs with dropped panels, (or $0.125l$ for slabs without dropped panels), on each side of the center line of columns, the full reinforcement required for negative moment will be provided in the top face of the slab. The full reinforcement for positive moment in the middle strip shall extend in the bottom face of the slab to a point at least $0.3l$ on either side of the panel

center line, and at least 50 per cent of it shall extend to points $0.325l$ on either side of the panel center line for slabs with dropped panels, or $0.35l$ for slabs without dropped panels.

1010: *Arrangement of Reinforcement—Four-Way System:*

(a) For direct bands, all provisions governing the placing of steel in column strips in two-way systems apply as well to the direct bands in four-way systems.

(b) For diagonal bands, at least four-tenths of the area of steel required at the section for positive moment shall be of such length and so placed as to reinforce the negative moment section at the two diagonally opposite column heads. These bars and any other bars for negative reinforcement shall extend into the adjoining panel to points at least $0.4l$ beyond a line drawn through the column center perpendicular to the direction of the band. The straight bars for positive moment in the diagonal bands shall not be shorter than the longer straight bars in the direct bands.

(c) For negative moment in the middle strip, the required steel shall extend not less than $0.25l$ on either side of the column center line.

1011: *Wall and Other Irregular Panels:*

(a) In wall panels and other panels in which the slab is non-continuous on one edge, the maximum positive moments on the principal design sections parallel to the discontinuous edge (reinforcement perpendicular to that edge) shall be increased by 25 per cent.

(b) The positive moment reinforcement perpendicular to the discontinuous edge shall extend to this edge and have an embedment of at least 6 in. in spandrel beams or columns. All negative moment reinforcement shall be bent or hooked at spandrel beams or columns to provide adequate bond resistance.

(c) At the wall or discontinuous edge the negative moment in the column strip shall be taken as not less than 90 per cent and in the middle strip not less than 62½ per cent of the corresponding moments for a normal interior panel as given in the table of Sec. 1003(b).

(d) Where there is a beam or a bearing wall at the center line of columns in the interior portion of a continuous flat slab, the negative moment at the beam or wall line in the middle strip perpendicular to the beam or wall shall be taken as 30 per cent greater than the negative moment specified in Sec. 1003(b) for a middle strip. The half column strip adjacent and parallel to and lying on either side of the beam or wall shall be designed to resist moments at least one-fourth of those specified in Sec. 1003(b) for a column strip. The beam or wall in such cases shall be designed to carry a uniformly distributed load equal to one-fourth of the panel load on either side in addition to the loads directly imposed upon it.

1012: *Panels With Marginal Beams:*

(a) In panels having marginal beam on one edge or on each of the two adjacent edges, the beam shall be designed to carry at least the load

superimposed directly upon it, exclusive of the panel load. A marginal beam which has a depth greater than 1½ times the minimum slab thickness, shall be designed to carry, in addition to the load superimposed directly upon it, a uniformly distributed load equal to at least ¼ of the total live and dead load for which the adjacent panel or panels are designed. Slabs supported by marginal beams on opposite edges shall be designed as freely supported slabs for the entire load.

(b) The half column strip adjacent to and parallel with marginal beams, having a depth not greater than 1½ times the minimum slab thickness, shall be designed to resist half the moment specified for a full column strip.

(c) In wall panels having exterior columns where brackets, (the faces of which make an angle with the face of the column, projected upward, of not more than 45 deg.) are used in place of capitals, the value of (c) in the direction in which the bracket extends may be taken as twice the distance from the center of the column to a point where the structural portion of the bracket is 1½ in. thick, and averaged with the value of (c) for an interior column capital in the computations for moment in formula (19). The value of (c) for column strips parallel and adjacent to a non-continuous edge of a slab where either no marginal beam is used, or where the beam used is not deeper than 1½ times the minimum slab thickness, should be taken as equal to the width of the wall column if no bracket is used in this direction.

(d) The value of (c) for column strips parallel and adjacent to marginal beams having a depth greater than the thickness of the slab at the wall columns, shall, if no bracket is used in this direction, be taken as equal to the width of the wall column plus twice the difference between the depth of the beam and the depth of the slab through the dropped panel. This value of c is to be used in calculating the $-M_c$ and $+M_c$ for the half column strip parallel and adjacent to the marginal beams only. This half column strip should be designed to resist a moment at least one-fourth as great as that specified for a column strip in the Table of Moments.

(e) It shall be permissible to omit the dropped panels at wall columns provided the design complies with the requirements of Section 1003-b and 1006-a for slabs without dropped panels.

1013: *Openings in Flat Slabs:*

(a) Openings of any size may be cut through the floor in the area common to two intersecting middle strips, provided the total positive and negative resisting moments be maintained as required in Sec. 1003(b) and that these total positive and total negative moments be redistributed between the remaining principal design sections to meet the new conditions.

(b) In any area common to two column strips, not more than one opening shall be allowed and the greatest dimension of such an opening shall not exceed $1/20l$.

(c) In any area common to one column strip and one middle strip, openings shall not interrupt more than one-quarter of the bars in either

strip and the equivalent of the bars so interrupted shall be provided by extra steel on both sides of the opening.

(d) Any opening larger than described above shall be completely framed on all sides with beams to carry the loads to the columns.

CHAPTER 11.

REINFORCED-CONCRETE COLUMNS AND WALLS.

1101: *Limiting Dimensions:*

(a) Unless designed as long columns under the provisions of Sec. 1108, reinforced-concrete columns shall not be longer than eleven times the least lateral dimension. Principal columns in buildings shall have a minimum diameter or thickness of 12 in. Posts that are not continuous from story to story shall have a minimum diameter or thickness of 6 in.

1102: *Unsupported Length of Columns:*

(a) The unsupported length of reinforced-concrete columns shall be taken as:

(1) In flat-slab construction the clear distance between the floor and under side of the capital;

(2) In beam-and-slab construction, the clear distance between the floor and the under side of the shallowest beam framing into the column at the next higher floor level;

(3) In floor construction with beams in one direction only, the clear distance between floor slabs;

(4) In columns supported laterally by struts or beams only, the clear distance between consecutive pairs (or groups) of struts or beams, provided that to be considered an adequate support, two such struts or beams shall meet the column at approximately the same level and the angle between the two planes formed by the axis of the column and the axis of each strut respectively is not less than 75 deg., nor more than 105 deg.

(b) When reinforced-concrete brackets are used at the junction of beams or struts with columns, the clear distance between supports may be considered as reduced by the depth of the bracket, provided the width of the bracket is at least equal to that of the beam and not less than one-half of the column.

1103: *Design of Spiral Columns:*

(a) The permissible axial load on columns reinforced with longitudinal bars and closely spaced spirals enclosing a circular core, shall not be greater than that determined by formula (22).

$$P = A_c\ [1 + (n - 1)\ p]\ f_c \qquad \text{(22)}$$

in which A_c is the area within the outer circumference of the spiral hooping, and the values of f_c are as given in Sec. 306, or as may be found for intermediate values of p by interpolation, or in general, by the formula,

$$f_c = [300 + (0.10 + 4p) f'_c\ ] \qquad \text{(22a)}$$

(b) The longitudinal reinforcement shall consist of at least six bars of minimum diameter of ½ in., and of an effective cross sectional area not less than 0.01, nor more than 0.06 of that of the core. The number of longitudinal bars concentrated in the ring at the periphery of the core shall be governed by the spacing requirements of Section 504-a. If all the bars cannot be placed at the periphery of the core, the bars within shall be stayed at intervals of 24 in., and shall not be nearer to the outer ring than two-tenths times the core diameter. When the ratio of reinforcement in a spirally reinforced column is greater than 0.04, special placing drawings illustrating the proper distribution of steel shall be submitted with the detail plans. Splices in longitudinal reinforcement shall provide a lap of at least 24-bar diameters for deformed bars, and 30 diameters for plain bars.

(c) The ratio of the spiral reinforcement shall be not less than one-fourth the ratio of the longitudinal reinforcement. Spiral reinforcement shall consist of evenly spaced continuous spirals held firmly in place and true to line by at least three vertical spacer bars. At the ends of all spirals and at points of splice, the outside diameter shall be maintained. The spacing of the spirals shall not be greater than one-sixth of the diameter of the core and in no case more than 3 in.

(d) Reinforcement shall be protected everywhere by a covering of concrete cast monolithic with the core which shall have a minimum thickness of 1½ in.

1104: *Design of Columns with Lateral Ties:*

(a) The permissible axial load on columns reinforced with longitudinal bars and separate lateral ties shall not be greater than that determined by formula (23):

$$P = 0.225 f'_c A_g [1 + (n - 1)p] \quad \ldots\ldots\ldots\ldots\ldots\ldots (23)$$

(b) The ratio of longitudinal reinforcement shall not be less than 0.005 nor shall the ratio considered in the calculations be more than 0.02 of the total area of the column. The longitudinal reinforcement shall consist of not less than four bars of minimum diameter of ⅝ in., placed with clear distance from the face of the column not less than 2 in., nor more than 3 in. Splices in longitudinal reinforcement shall provide a lap of at least 24-bar diameters for deformed bars, and 30 diameters for plain bars.

(c) Lateral ties shall be at least ¼ in. in diameter spaced not more than 12 in. apart. In columns of rectangular section, cross ties shall be arranged to afford support to the vertical bars at intervals not greater than the shorter side of the section, but such interval need not be less than 12 in. in any case.

1105: *Bending in Columns:*

(a) The bending moments in interior and exterior columns shall be determined on the basis of loading conditions and end restraint, and shall be provided for in the design.

(b) In flat-slab construction, the least dimension of the column shall be not less than one-fifteenth of the average center to center span, nor less

than 16 in. For known eccentric loads or unequal spacing of columns, computations of moments shall be made accordingly. Wall columns in flat-slab construction shall be designed to resist a bending moment of $Wl/35$. Any counter moment due to the weight of the structure that projects beyond the column center line may be deducted from the moment computed as just described. Resistance to the bending moments shall be divided between the columns immediately above and below in direct proportion to the values of their ratios of I/h (see Sec. 709 and 1102).

(c) The recognized methods shall be followed in calculating the stresses due to combined axial load and bending. The column section shall not be less than that required where axial load alone is considered. The limiting combined unit stresses shall be as follows:

(1) Columns with spiral reinforcement,
$$[300 + (0.10 + 4p)f'_c] + 0.15f'_c$$
(2) Columns with lateral ties $0.3f'_c$. The total amount of reinforcement considered in the computations shall not be more than 4 per cent of the total area of the column.

(3) Tension in longitudinal reinforcement due to bending on the column shall not exceed 16,000 lb. per sq. in.

(d) Where the allowable unit stress in columns is increased (to provide for combined axial load and bending) and wind loads are also added, the total shall still come within the allowable values specified for wind loads in Sec. 604.

1106: *Composite Columns:*

(a) The permissible load on composite columns in which a structural steel or cast-iron column is thoroughly encased in a concrete core reinforced with not less than 0.02 nor more than 0.04 longitudinal reinforcement in the form of bars arranged at the periphery of the core, nor less than 0.01 of spiral reinforcement meeting the requirements for spirals of Sec. 1103 (c), shall be based on a certain unit stress for the steel or cast-iron section plus a unit stress of $0.25f'_c$ on the net area of the concrete within the outer circumference of the spiral hooping enclosing the core. The longitudinal and spiral reinforcement ratios stated shall be based on the total core area enclosed within the outer circumference of the spiral hooping.

(b) The unit compressive stress on the steel section shall not exceed 15,000 lb. per sq. in. Where the steel section is required to carry construction or other loads prior to its encasement in concrete, the stress shall not exceed that given by formula (24).

$$f_r = \frac{18{,}000}{1 + \dfrac{h^2}{18{,}000\, R^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (24).$$

(c) The unit stress on the cast-iron section shall not exceed 9,000 lb. per sq. in. Where the cast-iron section is required to carry construction, or other loads prior to its encasement in concrete, the stress shall not exceed that given by formula (25).

$$f_r = 12{,}000 - 60\frac{h}{R} \quad \ldots\ldots\ldots\ldots (25).$$

(d) The unit stress on the longitudinal reinforcement shall be $0.25nf'_c$.

(e) The diameter of the cast-iron section shall not exceed one-half the diameter of the core, nor shall its total area exceed 12 per cent of the core area, (area included within outer circumference of the spiral hooping). The dimension of the structural steel section shall be such as to provide at least 3 in. between the spiral and the corners of the section and its area shall not exceed 12 per cent of the core area. Metal columns shall be accurately milled at splices and positive provision shall be made for alignment of one column above another. The spiral reinforcement shall be not less than 0.01 of the volume of the core, and shall conform in quality, spacing, and other requirements to the provisions for spirals in Sec. 1103(c).

(f) In composite columns, provision shall be made at the base to transfer the load from the middle section at safe unit stresses in accordance with Section 1205. The base of the metal section shall be designed to transfer the load from the entire composite column to the foundation, or it may be designed to transfer the load from the metal section only, provided it is so placed in the pier or pedestal as to leave ample section of concrete above the base for the transfer of the load from the reinforced-concrete section of the column by means of bond on the vertical reinforcement, and by direct compression from the concrete. At the top of the metal section, provision shall be made to receive the full load to be transferred to the metal section at this point. At points in the structure below this, where the load on the metal section is increased, positive means, consisting of cast or built-up brackets rigidly attached to the metal section, shall be provided to receive the increase in load.

(g) Ample section of concrete and continuity of reinforcement shall be provided at the junction with beams or girders. The area of the concrete between the spiral and the metal column shall be not less than that required to carry the total floor load of the story above on the basis of a stress in the concrete of $0.35f'_c$, unless special brackets are arranged on the metal column to receive directly the beam or slab load.

1107: *Combination Columns:*

(a) Structural steel columns of any rolled or built-up section wrapped with the equivalent of No. 8 U. S. standard gage wire spaced 4 in. on center and encased in concrete not less than 2 in. thick over all of metal except rivet heads and connections will be permitted to carry a load equal to $(1 + A_c/100A_s)$ times permissible load for unencased steel columns.

(b) The permissible load for unencased steel columns shall be determined by formula (24), provided the structural steel column acting independently of the concrete shall have sufficient capacity to carry all dead loads which will be placed thereon, and provided the quality of the concrete is such that it shall show a compressive strength of at least 2,000 lb. per sq. in. at 28 days when tested in accordance with Sec. 201 (c).

1108: *Long Columns:*

(a) The permissible working load on the core in axially loaded spiral or composite columns which have a length greater than 50 times the least radius of gyration of the column core ($50R$) shall not be greater than that determined by formula (26).

$$\frac{P'}{P} = 1.50 - \frac{h}{100R} \qquad \text{(26)}$$

(b) The permissible working load on axially-loaded tied columns, which have a length greater than 40 times the least radius of gyration of the column section ($40R$), shall not be greater than that determined by formula (26a).

$$\frac{P'}{P} = 1.33 - \frac{h}{120R} \qquad \text{(26a)}$$

(c) The radius of gyration of a column shall be computed from the concrete area used in design and the transformed section of the longitudinal steel area; that is, the actual area of steel multiplied by n.

1109: *Monolithic Walls:*

(a) Reinforced-concrete *bearing* walls shall have a thickness of at least one twenty-fifth (1/25) of the unsupported height or width, provided, however, that approved buttresses, built-in columns, or piers designed to carry all the vertical loads, may be used in lieu of greater thicknesses. Working compressive stresses in such walls shall not exceed $0.0625f'_c$ when the wall is 25 times the thickness in height, increasing proportionately to $0.125f'_c$ when the wall is 15 times the thickness or less in height.

(b) The lateral support for such walls shall consist of a fire-resistive floor when the framing is on one side of the wall only, but may be of a fire-resistive or of a non-fire-resistive type where framing is on both sides of the wall, provided that for residences, wood-frame construction properly tied may be used as support.

(c) In fire-resistive buildings, reinforced-concrete *bearing* walls shall have a thickness at least equal to the values shown in the table of minimum wall thicknesses given at the end of this section, except that exterior basement walls shall not be less than 8 in. thick. (See table page 243.)

(d) In fire-resistive buildings, bearing walls shall be reinforced with an area of steel in each direction, vertical and horizontal, at least equal to 0.0025 times the cross-sectional area. Walls 8 in. or more in thickness shall have half the steel at each face of the wall. The bars shall not be farther apart in either direction than 18 in., regardless of whether the

steel is disposed in one or two layers, nor shall less than the equivalent of ⅜-in. round bars be so used. The vertical steel shall not be relied on to carry load unless tied and arranged as in reinforced-concrete columns.

(e) All bearing walls shall be designed for any lateral pressure to which they are subjected. Eccentric loads and wind stresses shall be fully provided for. In such designs, the stresses for flexure as given in Sec. 306 shall govern.

(f) In non-fire-resistive buildings, exterior bearing walls may be of reinforced concrete, subject to the provisions of this section, when increased 50 per cent in thickness over those referred to in (c). In such walls, the amount of reinforcement in each direction, horizontal and vertical, shall be at least 0.002 times the cross-sectional area. The steel shall be distributed half to each face of the wall with a maximum bar spacing of 24 in.

(g) In buildings of skeleton construction, panel or other walls supported on the structural frame shall not be less than 5 in. thick, nor less than one-thirtieth (1/30) of the horizontal distance between columns, cross walls, or equivalent anchorage. Such walls shall be reinforced in the same manner as bearing walls in fireproof buildings, (see (d) above).

(h) Stairway and elevator enclosures in all classes of buildings may be built of reinforced concrete, when the wall thicknesses are in accordance with the requirements of (c) and (g) and reinforcement in accordance with (d).

MINIMUM WALL THICKNESS, IN INCHES, IN STORY INDICATED.

No. of Stories	Basement	1st	2nd	3rd	4th	5th	6th	7th	8th	9th	10th
1	6	6	..	..	..	..	..	..	..	..	..
2	7	6	6	..	..	..	..	..	..	..	..
3	8	7	7	6	..	..	..	..	..	..	..
4	8	8	7	7	6	..	..	..	..	..	..
5	9	8	8	7	7	6	..	..	..	..	..
6	9	9	8	8	7	7	6	..	..	..	..
7	10	9	9	8	8	7	7	6	..	..	..
8	10	10	9	9	8	8	7	7	6	..	..
9	12	10	10	9	9	8	8	7	7	6	..
10	12	12	10	10	9	9	8	8	7	7	6

CHAPTER 12.

FOOTINGS.

1201: *Loads:*

(a) Footings resting directly on soil or on piles shall be proportioned as to area or number of piles on the basis of the total column load plus the weight of the footing itself. For computations of moments and shears, an upward reaction per unit area or per pile shall be based on the total column load (not including the weight of the footing itself) divided by the area or by the number of piles.

1202: *Sloped or Stepped Footings:*

(a) Footings in which the thickness has been determined by the requirements for shear as specified in Sec. 808, may be sloped or stepped between the critical section and the edge of the footing, provided that the shear on no section outside the critical section exceeds the value specified, and provided further that the thickness of the footing above the reinforcement at the edge shall not be less than 6 in. for footings on soil, nor less than 12 in. for footings on piles. Sloped or stepped footings shall be cast as a unit.

1203: *Bending in Footings:*

(a) The critical section for bending in a concrete footing which supports a concrete column or pedestal, shall be considered to be at the face of the column or pedestal. Where steel or cast-iron column bases are used, the moment in the footing shall be computed at the middle and at the edge of the base; the load shall be considered as uniformly distributed over the column or pedestal base.

(b) The bending moment at the critical section in a square footing supporting a concentric square column, shall be computed from the load on the trapezoid bounded by one face of the column, the corresponding outside edge of the footing, and the portions of the two diagonals. The load on the two corner triangles of this trapezoid shall be considered as applied at a distance from the face equal to six-tenths of the projection of the footing from the face of the column. The load on the rectangular portion of the trapezoid shall be considered as applied at its center of gravity. The bending moment is expressed by formula (27).

$$M = \frac{w}{2}(a + 1.2c)\,c^2 \quad \dots\dots\dots\dots \quad (27)$$

(c) For a round or octagonal column, the distance a shall be taken as equal to the side of a square of an area equal to the area enclosed within the perimeter of the column.

1204: *Shearing and Bond Stresses:* See Sec. 808, also Sec. 901 to 904.

1205: *Transfer of Stress at Base of Column:*

(a) The compressive stress in longitudinal reinforcement at the base of a column shall be transferred to the pedestal or footing by dowels. There shall be at least one dowel for each column bar, and the total sectional area of the dowels shall not be less than the sectional area of the longitudinal reinforcement in the column. The dowels shall extend into the column and into the pedestal or footing not less than 30 diameters of the dowel bars for plain bars, or 24 diameters for deformed bars.

(b) The permissible compressive unit stress on top of the pedestal or footing directly under the column shall be not greater than that determined by formula (28).

$$r_a = p_a \sqrt[3]{\frac{A}{A'}} \quad \dots\dots\dots\dots \quad (28)$$

(c) The value of p_a shall not exceed 0.25 f'_c for plain concrete. When lateral reinforcement in the form of spiral or hoops is provided, the value of p_a for the area within the spiral may be increased $(1 + 2.5np')$ times that for plain concrete, but no area outside the outer face of the spiral shall be considered. Where piers are designed as columns, the value of p_a shall be computed by the proper column design formula.

(d) In no case shall the total load computed by formula (28) be taken as greater than the load computed, using a stress equal to p_a, on the gross area of the pedestal, pier, or footing at a point below special reinforcing provided at the top.

(e) Where the loaded area is not central on the top of the pedestal pier, or footing, the total area A shall not be taken as greater than the area of the largest circle that can be drawn about the load as a center and lying entirely within the top of the pedestal, pier, or footing.

(f) Where lateral reinforcement is provided to increase the value of p_a, it shall extend to within 3 in. of the top of the pedestal, pier, or footing and to a depth equal to the diameter of the spiral, and the loaded area shall lie at the center of the spiral or hoops. The pitch of the spiral or the spacing of the hoops *in the clear* shall not be less than 2 in., nor more than 5 in. The designed pitch shall be maintained by at least four spacers securely fastened to each spiral turn or hoop. The ratio of lateral reinforcement shall not exceed 0.015.

(g) In sloped or stepped footings, A may be taken as the area of the top horizontal surface of the footing or as the area of the lower base of the largest frustum of a pyramid or cone contained wholly within the footing and having for its upper base the loaded area A', and having side slopes of 1 vertical to 2 horizontal.

1206: *Pedestals without Reinforcement:*

(a) The allowable compressive unit stress on the gross area of a concentrically loaded pedestal or on the minimum area of a pedestal footing shall not exceed $0.25f'_c$, unless reinforcement is provided and the member designed as a reinforced-concrete column.

(b) The depth of a pedestal or pedestal footing shall not be greater than three times its least width and the projection on any side from the face of the supported member shall not be greater than one-half the depth. The depth of a pedestal whose sides are sloped or stepped shall not exceed three times the least width or diameter of the section midway between the top and bottom. A pedestal footing supported directly on piles shall have a mat of reinforcing bars having a cross-sectional area of not less than 0.20 sq. in. per ft. in each direction, placed 3 in. above the top of the piles.

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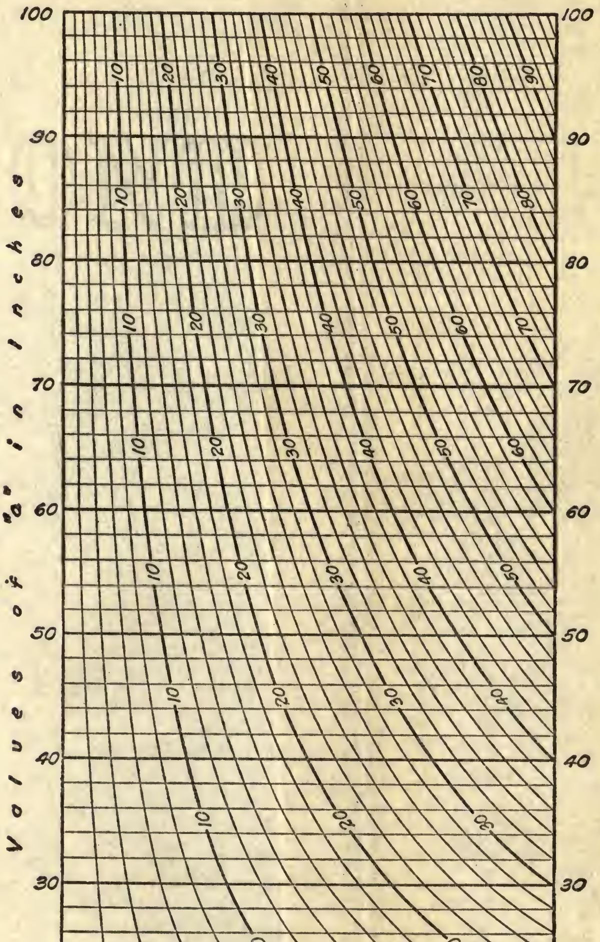
DIAGRAM 72.—READING CHART FOR DIAGRAMS 69, 70 AND 71



See instructions for use under Table 68.